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16. Abstract

This Interim Technical Bulletin recommends procedures for conducting Life Cycle Cost Analysis (LCCA) of pavements, provides detailed procedures to determine work zone user costs, and introduces a probabilistic approach to account for the uncertainty associated with LCCA inputs. The bulletin begins with a discussion of the broad fundamental principles involved in a LCCA. It discusses input parameters and presents simple examples of traditional LCCA in a pavement design setting. It discusses the variability and inherent uncertainty associated with input parameters, and provides recommendation on acceptable ranges for the value of time as well as discount rates. It explores the use of sensitivity analysis in traditional LCCA approaches. User costs are a combination of delay, vehicle operating costs, and crash costs. Each of these cost components are explored and procedures are presented to determine their value. Given the power and sophistication of today's computers and software, simulation techniques, such as Monte Carlo simulation, are recommended for incorporating variability associated with LCCA inputs into final results.

The Final Technical Bulletin will include procedures for developing a probabilistic approach to conduct LCCA, as well as actual case studies of the probabilistic approach compared to traditional analysis.

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EXECUTIVE SUMMARY

This Interim Technical Bulletin provides technical guidance and recommendations on "good practice" in conducting Life Cycle Cost Analysis (LCCA) in pavement design and introduce Risk Analysis, a probabilistic approach to describe the uncertainty inherent in the process. It is geared toward State highway agency (SHA) personnel with responsibility for conducting and/or reviewing LCCA of highway pavement designs.

It begins with a discussion of the broad fundamental principles involved in LCCA followed by presentation of the widely accepted procedures used in setting up and conducting LCC analysis. It discusses input parameters and presents examples of traditional LCCA in a pavement design setting. It goes on to provide a detailed rational highway capacity based approach for determining work zone user costs associated with alternative pavement design strategies.

It discusses the variability and inherent uncertainty associated with input parameters, and provides recommendations on acceptable ranges for a variety of parameters. It recommends the use of NPV to compare alternatives based on real (non inflated) dollars and real discount rates of between 3 and 5 percent. It also provides default dollar value ranges for the hourly value of time associated with user delay and dollar values associated with fatal and non-fatal injury highway crashes.

It explores the use of sensitivity analysis and introduces a risk analysis approach to account for the variability of inputs. It also includes discussion of supporting computer software and additional LCCA resource documents are included in the appendix.

This Interim Technical Bulletin deals specifically with the technical aspects of the long term economic efficiency implications of alternative pavement designs. It does not address equity issues, which are likely to be a highly significant consideration in any public investment decision. LCCA results, while a useful decision support tool, are not decisions in and of themselves. Frequently, the resulting LCCA results are less important than the analytical evaluation that such analysis fosters.

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LIST OF ABBREVIATIONS

AADT	Annual Average Daily Traffic
BAMS	Bid Analysis Management System
B/C	Benefit Cost Ratio
CFR	Code of Federal Regulation
DP-115	Demonstration Project 115
DOT	Department of Transportation
EUAC	Equivalent Uniform Annual Cost
FHWA	Federal Highway Administration
GAO	Government Accounting Office
HCM	Highway Capacity Manual
HMAC	Hot Mix Asphalt Concrete
HOT	High Occupancy Toll
HOV	High Occupancy Vehicle
IRI	International Roughness Index
IRR	Internal Rate of Return
ISTEA	Intermodal Surface Transportation Efficiency Act
LCC	Life Cycle Cost
LCCA	Life Cycle Cost Analysis
LTPP	Long Term Pavement Performance
NHS	National Highway System
NPV	Net Present Value
OIG	Office of Inspector General
OMB	Office of Management and Budget
PA-DOT	Pennsylvania Department of Transportation
PCCP	Portland Cement Concrete Pavement
SHA	State highway agency
VMT	Vehicle Miles of Travel
VOC	Vehicle Operating Cost
WZ	Work Zone

CHAPTER 1 - INTRODUCTION

1.1 PURPOSE

The purpose of this Interim Technical Bulletin is to provide technical guidance and recommendations on "good practice" in conducting Life Cycle Cost Analysis (LCCA) in pavement design and introduce Risk Analysis, a probabilistic approach to describe the uncertainty inherent in the process. The primary audience for this Interim Technical Bulletin is State highway agency (SHA) personnel with responsibility for conducting and or reviewing LCCA of highway pavements. This includes State pavement design engineers, pavement management engineers, as well as district or area supervisors with responsibility for selecting pavement type and rehabilitation strategies.

1.2 SCOPE

This Interim Technical Bulletin recommends specific procedures for conducting LCCA in pavement design and discusses the relative importance of LCCA factors on analysis results. In the interest of technical purity all relative LCCA factors are discussed even though not all elements have the same degree of influence on the final LCCA results. It begins with a discussion of the broad fundamental principles involved in LCCA followed by presentation of the widely accepted procedures used in setting up and conducting LCC analysis. It discusses input parameters and presents examples of traditional LCCA in a pavement design setting. It discusses the variability and inherent uncertainty associated with input parameters, and provides recommendations on acceptable ranges for a variety of parameters. It explores the use of sensitivity and introduces a risk analysis approach to account for the variability of inputs. It also includes discussion of supporting computer software. The appendix lists additional LCCA resource documents specific to pavement design.

While the issue of equity is a highly significant consideration in any public investment decision, it is not part of the economic efficiency issue. This Interim Technical Bulletin deals specifically with the technical aspects of the long term economic efficiency implications of alternative pavement designs.

LCCA results, while a useful decision support tool, are not decisions in and of themselves. Frequently, the resulting LCCA results are less important than the analytical evaluation that such analysis fosters. As a result, SHAs are encouraged to conduct LCCA in support of all major investment decisions.

1.3 APPLICATION

Fundamental principals of economic analysis have broad application. In general, the concepts presented in this Interim Technical Bulletin are generic and may be applied to areas other than pavements. For example, LCCA may be applied to establish funding levels, allocate resources between program areas, and prioritize project selection. This Interim Technical Bulletin addresses project level design and rehabilitation strategy selection.

1.4 LEVEL OF DETAIL

The relative influence of individual LCC factors on analysis results may vary from major, to minor, to insignificant. The analyst should assure that the level of detail incorporated in a LCCA is consistent with the level of investment decision under consideration. As more and more cost factors are incorporated in an LCCA there comes a point of diminishing returns. For example, slight differences in future costs have a marginal effect on discounted present worth. Inclusion of such factors as this unnecessarily complicates the analysis without providing tangible improvement in analysis results. Frequently, inclusion of all factors in every analysis is not productive. The difficulty in capturing some costs makes omitting them the more prudent choice - particularly when the impact on the LCCA results is marginal at best.

In conducting a LCCA, analysts should evaluate all factors for inclusion and explain the rational for not including eliminated factors. Such explanations make analysis results more supportable when scrutinized by critics who are not pleased with the analysis outcome. This Interim Technical Bulletin does not provide guidance on determining the appropriate extent of LCCA on specific projects.

1.5 LCCA DRIVING FORCES

The FHWA position on pavement related LCCA has its roots in the Intermodal Surface Transportation Efficiency Act (ISTEA) of 1991, which specifically required consideration of

"The use of life-cycle costs in the design and engineering of bridges, tunnels, or pavement" in both Metropolitan and Statewide Transportation Planning. Additional direction came in January 1994 with Executive Order 12893, "Principles for Federal Infrastructure Investments," which requires that benefits and costs be systematically analyzed in making infrastructure investment decisions, and that they be measured and discounted over the full life cycle of each project. Further, an Office of Inspector General/Government Accounting Office (OIG/GAO) 1994 Highway Infrastructure report on cost comparison of asphalt versus concrete pavements reviewed in FHWA Region 4 states, made specific recommendations on FHWA's need to provide additional technical guidance on LCCA.

Finally, the NHS Designation Act of 1995 specifically requires that the Secretary establish a program that requires States to conduct life-cycle costs analysis on NHS projects where the cost of a usable project segment equals or exceeds $25 million. Implementing guidance on NHS LCCA requirements was distributed to FHWA field offices in an April 19, 1996 memorandum from the FHWA Executive Director, Anthony Kane. The implementing guidance did not prescribe specific LCCA procedures but rather focused on the use of "good practice".

The NHS Designation Act of 1995 also required SHAs to perform Value Engineering Analysis on the same high cost NHS projects. The Value Engineering provisions were implemented in 23 CFR Part 627 promulgated in the *Federal Register* Vol. 62, No. 31, page 6866, on February 14, 1997. The requirements in 23 CFR Part 627 took effect on March 17, 1997.

1.6 GENERAL DEFINITIONS

Some of the more general definitions used in this technical bulletin are listed below. Other definitions are provided in the sections where they are addressed.

Life Cycle Cost Analysis (LCCA), as defined in Section 303, *Quality Improvement* of the National Highway System (NHS) Designation Act, is "a process for evaluating the total economic worth of a usable project segment by analyzing initial costs and discounted future cost, such as maintenance, reconstruction, rehabilitation, restoring, and resurfacing costs, over the life of the project segment" A ***useable project*** segment is defined as a portion of a highway that, when completed, could be opened to traffic independent of some larger overall project.

In simpler terms LCCA is an analysis technique that supports more informed and, hopefully, better investment decisions. It builds on some well founded principles of economic analysis that have been used in evaluating highway and transit programs for years with a slightly stronger focus on the longer term. It incorporates discounted long term agency, user, and other relevant costs over the life of a highway or bridge to identify the best value for investment expenditures (i.e. the lowest long term cost that satisfies the performance objective being sought). It can be applied to a wide variety of investment related decision levels to evaluate the economic worth of various designs, projects, alternatives, or system investment strategies to get the best return on the dollar.

Pavement Design is defined under 23 CFR section 500.203 as "... a project level activity where detailed engineering and economic considerations are given to alternate combinations of subbase, base, and surface material which will provide adequate load carrying capacity. Factors that are considered include: materials, traffic, climate, maintenance, drainage and life cycle costs."

User Costs are costs incurred by highway users traveling on the facility, and the excess costs incurred by those who can not use the facility due to either agency or self imposed detour requirements. User costs typically are an aggregation of three separate components: Vehicle Operating Costs (VOC), Crash Costs, and User Delay Costs. Each of these cost components will be discussed in greater detail in Chapter 3.

Deterministic Approach to LCCA is the application of procedures and techniques without regard for the variability of the inputs. The primary disadvantage of this "traditional" approach is that it does not account for the variability associated with the LCCA input parameters.

This Interim Technical Bulletin advocates a ***Risk Analysis Approach*** in the characterization of uncertainty. This approach combines probability descriptions of analysis inputs with computer simulations to generate the entire range of outcomes as well as the likelihood of occurrence.

There are several economic indicators available to the analyst. The most common include, Benefit/Cost (B/C) Ratios, Internal Rate of Return (IRR), Net Present Value (NPV) and Equivalent Uniform Annual Costs. Many of these indicators are thoroughly discussed in the Federal Government's Office of Management and Budget Circular A-94.(1)

Benefit/Cost Analysis or Ratio represents the net discounted benefits of an alternative divided by net discounted costs. B/C ratios greater than one indicate benefits exceed cost. The B/C ratio is generally not recommended for pavement analysis due to the difficulty in sorting out benefits and costs for use in developing B/C ratios.

Internal Rate of Return (IRR) represents the discount rate necessary to make discounted cost and benefits equal. While the internal rate of return does not generally provide an acceptable decision criterion, it does provide useful information, particularly when budgets are constrained or there is uncertainty about the appropriate discount rate.

Net Present Value (NPV), sometimes called Net Present Worth, is the discounted monetary value of expected net benefits (i.e., benefits minus costs). Net present value is computed by assigning monetary values to benefits and costs, discounting future benefits and costs using an appropriate discount rate, and subtracting the sum total of discounted costs from the sum total of discounted benefits.

Discounting benefits and costs transforms gains and losses occurring in different time periods to a common unit of measurement. Programs with positive net present value increase social resources and are generally preferred. Programs with negative net present value should generally be avoided. There is fairly strong agreement in the literature that NPV/NPW is the economic efficiency indicator of choice. The basic formula for computing NPV is as follows.

$$NPV = \text{Initial Cost} + \sum_{k=1}^{n} \text{Rehab Cost}_k \left[\frac{1}{(1+i)^{n_k}} \right]$$

where: i = discount rate
n = number of years into future

NPV computations are discussed in more detail in Section 2.6.

Equivalent Uniform Annual Costs (EUAC) represents the NPV of all discounted cost and benefits of an alternative as if they were to occur uniformly throughout the analysis period. EUAC is a particularly useful indicator when budgets are established on an annual basis. The preferred method of determining EUAC is to first determine the NPV and then convert it to EUAC using the following formula:

$$EUAC = NPV \left[\frac{1(1+i)^n}{(1+i)^n - 1} \right]$$

where: i = discount rate
n = number of years into the future

Additional terms are defined as necessary as they occur in the body of the text.

1.7 COST ESTIMATES

Estimates of future costs and benefits can be made using constant or *nominal* dollars. *Constant* dollars, often called "real" dollars, reflect dollars with the same or constant purchasing power over time. In such cases the cost of performing an activity would not change as a function of the future year in which it will be accomplished. For example, if hot mix asphalt concrete (HMAC) were to cost $20/ton today, HMAC cost estimates for future year work would also be based on $20/ton. *Nominal* dollars, on the other hand, reflect dollars that fluctuate in purchasing power as a function of time. They are normally used to fold in future general price rises due to anticipated inflation. When using *nominal* dollars the estimated cost of an activity would change as a function of the future year in which it is accomplished. In this case, if HMAC were to cost $20/ton today and inflation were estimated at 5%, HMAC cost estimates for one year from today would be $21/ton.

While LCCA can be conducted using either *constant* or *nominal* dollars, there are two cautions. First, in any given LCCA, *constant* and *nominal* dollars cannot be mixed in the same analysis (i.e. all costs must be in either *constant* dollars or all costs must be in *nominal* dollars). Second, the discount rate (discussed below) selected must be consistent with the dollar type used (i.e. use *constant* dollars and discount rates or *nominal* dollars and *nominal* discount rates. Good/best practice suggests that LCCA should be conducted using *constant* dollars and *real* discount rates. This combination eliminates the need to estimate and include an inflation premium for both cost and discount rates.

1.8 DISCOUNT RATES

1.8.1 Nominal Verses Real

Similar to costs, discounts rates used in LCCA can be either real or nominal. *Real* discount rates reflect the true time value of money with no inflation premium and should be used in conjunction with non-inflated dollar cost estimates of future investments. *Nominal* discount rates include an inflation component and should only be used in conjunction with inflated future dollar cost estimates of future investments. The same caveats as to mixing real dollar cost and nominal discount rates and vice versa noted above applies. Further discussion of the *real verses nominal* dollar and discount rates issue can be found in the Office of Management and Budget's (OMB) Circular A-94 and in the annual updates of Appendix A to the Circular.

1.8.2 Values to Use

Discount rates can have a significant influence in the analysis result. LCCA should use a reasonable discount rate that reflects historical trends over long periods of time. Data on the historical trends over very long periods indicate that the real time value of money is approximately 4%.

In the Public Sector, since investment resources come from John Q. Public, either in the form of taxes or user fees, the discount rate used needs to be consistent with the opportunity cost of the public at large. One conservative indicator of the opportunity cost of money for the public at large is the super safe U.S. Government Treasury Bills. Figure 1. 1 reflects the historical trend of yields on 10-year Treasury notes. The upper curve reflects the nominal rate of return while the lower curve represents the inflation adjusted real rate of return. Over the period March 1991 through August 1996, the real rate of return ranges somewhere between 3 to 5 % and averages somewhere close to 4%.

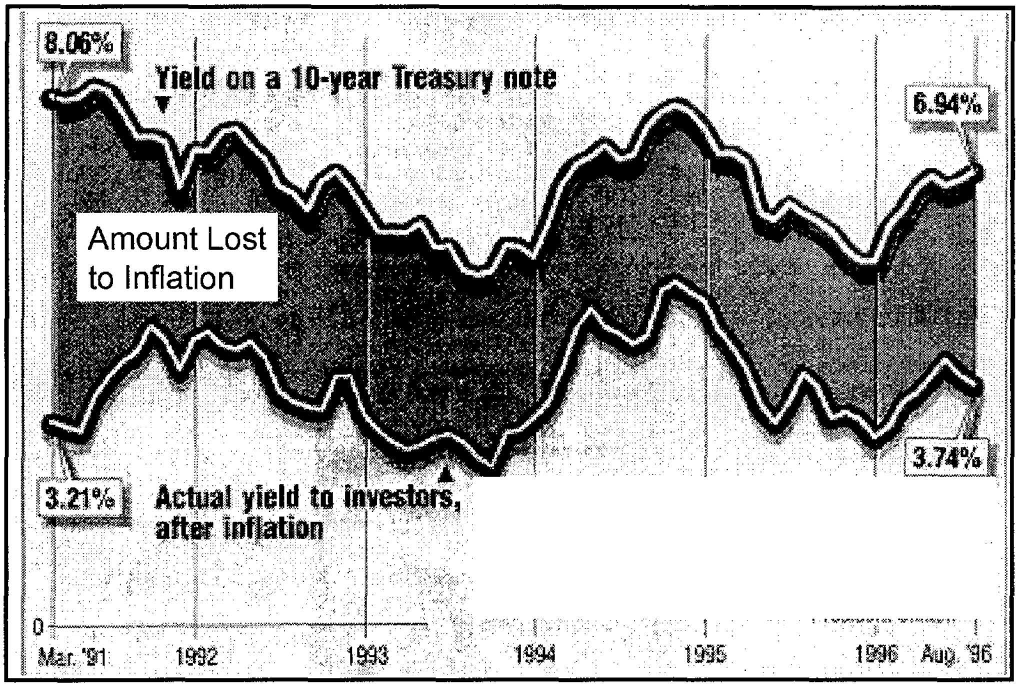

Figure 1. 1 Historical Trends on 10-Year Treasury Notes.

The Treasury Department made its first offering of Inflation Protected Securities to the general public in the spring of 1997. The offering was very well received by the public, (there was more demand for the securities than the Treasury Department wanted to sell), at a yield of just over 3.5%.

In the National Pavement Design Review conducted by the FHWA in 1995 and 1996, the discount rates currently employed by SHAs in conducting LCCA in pavement design showed a distribution of values clustering in the 3-5% range.

Finally, Table 1. 1 shows recent trends in real discount rates for various analysis periods published over the last several years from annual updates to OMB Circular A-94. (1)

Table 1. 1 Recent Trends in OMB Real Discount Rates.

Year	Analysis Period				
	3	**5**	**7**	**10**	**30**
92	2.7	3.1	3.3	3.6	3.8
93	3.1	3.6	4.0	4.3	4.5
94	2.1	2.3	2.5	2.7	2.8
95	4.2	4.5	4.6	4.8	4.9
96	2.7	2.7	2.8	2.8	3.0
97	3.2	3.3	3.4	3.5	3.6
Average	3.0	3.3	3.4	3.6	3.8
Standard Deviation	0.7	0.8	0.8	0.8	0.8

Considering the above, Good/best practice suggests the use of a real discount rate, one that does not reflect an inflation premium, of 3 to 5 percent in conjunction with real/constant dollar cost estimates.(2)

1.9 STRUCTURED APPROACH

Analysts should work from formalized, objective, LCCA procedures incorporated within the overall pavement design process. Such procedures should be comprehensive enough to capture and evaluate the differences between competing pavement design alternatives and subsequent rehabilitation strategies. The design process should clearly identify when and at what level the LCCA should be performed as well as the scope and level of detail of such analysis. LCCA procedures should clearly identify the components and factors that are included along with supporting rationale for selected input values. LCCA input assumptions should be reasonable and conform to accepted practice and convention. LCCA should recognize the uncertainty associated with LCCA inputs and the implication of the uncertainty on LCCA results. As a minimum, LCCA should include a sensitivity analysis of LCCA results to variation in major LCCA inputs. SHAs are encouraged to incorporate a quantitative risk analysis approach in the treatment of uncertain inputs (see chapter 4).

CHAPTER 2 - LCCA PROCEDURES

This chapter identifies the procedural steps involved in conducting a life cycle cost analysis (LCCA). They include:

1. Establish alternative pavement design strategies for the analysis period
2. Determine performance periods and activity timing
3. Estimate agency costs
4. Estimate user costs
5. Develop expenditure stream diagrams
6. Compute net present value
7. Analyze results
8. Reevaluate design strategy

While the "steps" are generally sequential, the sequence can be altered to meet specific LCCA needs. Each step will be discussed in the following sections.

2.1 ESTABLISH ALTERNATIVE PAVEMENT DESIGN STRATEGIES FOR THE ANALYSIS PERIOD

The primary purpose of a LCCA is to quantify the long-term implication of initial pavement design decisions on the future cost of maintenance and rehabilitation activities necessary to maintain some pre-established minimum acceptable level of service over some specified time period. The combination of initial pavement design and necessary supporting maintenance and rehabilitation activities represent a Pavement Design Strategy while the time horizon over which future cost are evaluated is referred to as the Analysis Period. The first step in conducting a LCCA of alternative pavement designs is to identify the alternative pavement design strategies for the analysis period under consideration.

Analysis Period

LCCA analysis periods should be sufficiently long to reflect long-term cost differences associated with reasonable design strategies. The analysis period should generally always be longer than the pavement design period except in the case of extremely long lived pavements.

As a rule of thumb the analysis period should be long enough to incorporate at least one rehabilitation activity. The FHWA's September, 1996, Final LCCA Policy statement recommends an analysis period of at least 35 years for all pavement projects, including new or total reconstruction projects as well as rehabilitation, restoration, and resurfacing projects.

At times, a shorter analysis periods may be appropriate, particularly when pavement design alternatives are set up to buy time (say 10 years) until total reconstruction. Deviation from the recommended minimum 35-year analysis period may also be appropriate when slightly shorter periods could simplify salvage value computations. For example, if all alternative strategies would reach terminal serviceability at year 32, than a 32-year analysis would be quite appropriate.

Regardless of the analysis period selected, the analysis period used should be the same for all alternatives. Figure 2. 1 shows the analysis period for a pavement design alternative.

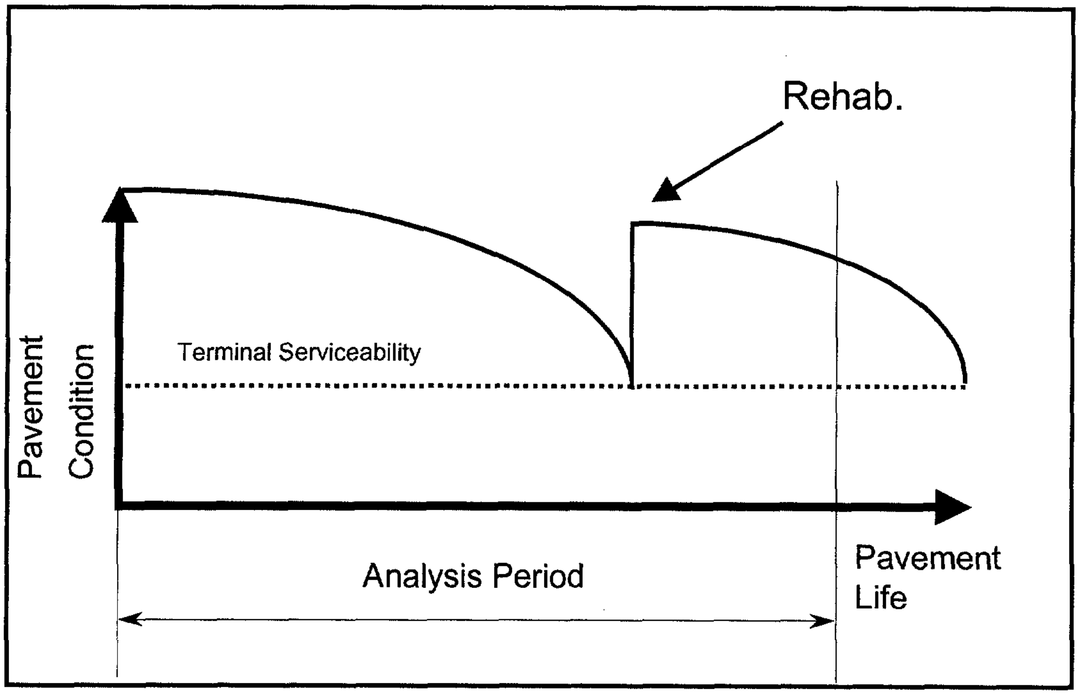

Figure 2. 1 Analysis Period for a Pavement Design Alternative.

Pavement Design Strategies

Typically, each design alternative will have an expected initial design life, periodic maintenance treatments, and possibly a series of rehabilitation activities. The scope, timing, and cost of these activities must be identified. Depending on the initial pavement design, State Highway Agencies employ a variety of rehabilitation strategies to keep the highway facilities in functional condition.(3, 4) For example, Table 2. 1 shows the Pennsylvania Department of Transportation's (PA-DOT) typical supporting maintenance and rehabilitation strategy for new, reconstructed and unbonded portland cement concrete pavements included in their LCCA procedures. PA-DOT's LCCA procedures also contain typical supporting strategies for new and reconstructed asphalt concrete pavements. Note that user cost requirements are also identified.

Table 2. 1 PA-DOT's Design Strategy for New, Reconstructed, and Un-Bonded Overlay.

Year	Treatment
5	Clean and seal 25% of longitudinal joints Clean and seal 5% of transverse joints. 0% for neoprene seals Seal coat shoulders if Type 1 paved shoulders.
10	Same as year 5
15	Clean and seal 25% of longitudinal joints Clean and seal 10% of transverse joints, 5% for neoprene seals Seal coat shoulders, if Type 1 paved shoulders
20	Concrete patch 5% of pavement area Spall repair 1% of transverse joints (5 sf/joint) Slab stabilization: minimum 25% of transverse joint Diamond grind 100% of pavement area Clean and seal all longitudinal joints, including shoulders Clean and seal all transverse joints, 7% for neoprene seals Seal coat shoulders, if Type I paved shoulders Maintenance and protection of traffic User delay
25	Clean and seal 25% of longitudinal joints Clean and seal 10% of transverse joints, 10% for neoprene seals Seal coat shoulders, if Type I paved shoulders
30	Concrete patch 2% of pavement area Clean and seal all joints with fiber asphalt membrane 60-#/sy leveling course 3.5-in. ID-2 or 4-in. ID-3/ID-2 overlay Saw and seal joints Type 7 paved shoulders Adjust all guide rail and drainage structures Maintenance and protection of traffic User delay
35	Seal coat shoulders
Note: The CPR strategy slated for year 20 can be moved to year 15 at the District's discretion. However, when doing this, the overlay at year 30 must be moved to year 25 and another overlay added at year 33.	

2.2 DETERMINE PERFORMANCE PERIODS AND ACTIVITY TIMING

Performance life for the initial pavement design and subsequent rehabilitation activities has a major impact on LCCA results. It directly affects the frequency of agency intervention on the highway facility, which in turn, affects agency cost as well as user costs during periods of construction and maintenance activities. SHAs can determine specific performance information for various pavement strategies through analysis of pavement management data and historical experience. Operational pavement management systems can provide the data and analysis techniques to evaluate pavement condition and performance and traffic volumes to identify cost-effective strategies for short and long-term capital projects and maintenance programs. Some SHAs develop performance lives based on the collective experience of their senior engineers.

Current efforts by FHWA to analyze pavement performance data collected as part of the SHRP Long Term Pavement Performance Program (LTPP) should provide an additional valuable resource to SHAs. In support of that effort, FHWA is also coordinating the development and wide distribution of the DataPave software program to make LTPP performance data directly available to the SHAs. Specific pavement performance information is also available in various pavement performance reports developed by SHAs such as Minnesota and Illinois just to mention a few.

Initial construction, maintenance, and rehabilitation work zone requirements directly impact highway user costs and must be estimated along with pavement strategy development. The frequency, duration, severity, and year of work zone requirement are critical factors in developing user costs for the alternative under consideration.

2.3 ESTIMATE AGENCY COSTS

Construction quantities and costs are directly related to the initial design and subsequent rehabilitation strategy. The first step in estimating agency costs is to determine construction quantities/unit prices. Unit prices can be determined from SHA historical data on previously bid jobs of comparable scale. State highway agency historical bid data as well as the Bid Analysis Management System (BAMS), if used by the SHA, are sources of data.

LCCA comparisons are always made between mutually exclusive competing alternatives. LCCA need only consider differential cost differences between alternatives. Costs common to all alternatives cancel out and are generally so noted and not included in LCCA calculations.

Agency costs include all costs incurred directly by the agency over the life of the project. They typically include initial preliminary engineering, contract administration, construction supervision and construction cost, as well as future routine and preventive maintenance, resurfacing and rehabilitation cost, and the associated administrative cost. Routine reactive type maintenance cost data are normally not available except on a very general, area wide, type cost per lane mile. Fortunately, routine reactive type maintenance costs generally are not very high due, primarily, to the relatively high performance levels maintained on major highway facilities. Further, SHAs that do report routine reactive type maintenance costs note little difference between most alternative pavement strategies. When discounted to the present, small reactive maintenance cost differences have negligible effect on NPV and can generally be ignored.

Agency costs also include maintenance of traffic cost and can include operating cost such as for interchange lighting and tunnel lighting and ventilation. Some times salvage value, the remaining value of the investment at the end of the analysis period, is included as a negative cost.

Salvage Value represents value of an investment alternative at the end of the analysis period. Salvage value can have two fundamental components associated with it: residual value and serviceable life.

Residual Value refers to the net value from recycling the pavement. The differential residual value between pavement design strategies is generally not very large, and, when discounted over 35 years, tends to have little impact on LCCA results.

Serviceable life represents the more significant salvage value component and is the remaining life in a pavement alternative at the end of the analysis period. It is used primarily to account for differences in remaining pavement life between alternative pavement design strategies at the end of the analysis period. For example, over a 35 year analysis, alternative A reaches terminal serviceability at year 35, while alternative B requires a 10-year design rehabilitation at year 30. In this case the serviceable life of alternative A at year 35 would be 0, as it has reached it's terminal serviceability. On the other hand, Alternative B s receives a 10-year design rehabilitation at year 30, and will have 5 years of serviceable life at year 35, the year the analysis terminates. The value of the serviceable life of alternative B at year 35 could be calculated as a percent of design life remaining at the end of the analysis period (5 of 10 years or 50%) multiplied by the cost of alternative B's rehabilitation at year 30.

Sunk Costs represent a special category that is irrelevant to the decision at hand. Analyst should be careful not to include them in LCCA. An illustrative example may serve best in understanding the concept.

> An individual places a $10 non-refundable down payment on a $100 camera at store - A. Before picking up the camera, the individual finds an identical camera on sale at Store - B for $80. From an economic efficiency perspective, from which store should the individual purchase the camera? What bearing does the $10 deposit have on the decision?

The $10 down payment is a sunk cost and is irrelevant to the decision. The decision comes down to paying Store - A the $90 balance for the camera or to paying Store - B $80 for an identical camera.

Not all cases of sunk cost are this clear and analysts need to take care to guard against including them in LCCA. An example more specific to pavement design might involve the reluctance of a designer to select an alternative with a much lower life-cycle cost because it would mean "wasting" the money previously spent on developing final plans for a clearly inferior alternative.

2.4 ESTIMATE USER COSTS

In the simplest sense user costs are costs incurred by the highway user over the life of the project. In LCCA, highway user costs of concern are the differential costs incurred by the motoring public between competing alternative highway improvements and associated maintenance and rehabilitation strategies over the analysis period. In the pavement design arena, the user costs of interest are further limited to the differences in user costs resulting from differences in long term pavement design decisions and the supporting maintenance and rehabilitation implications. User costs are an aggregation of three separate cost components; vehicle operating costs (VOC), user delay costs, and crash costs.

2.4.1 Normal Operations Verses Work Zones

In the LCCA of pavement design alternatives, there are user costs associated with both normal operations and work zone operations. The normal operations category reflects highway user costs associated with use of a facility during periods free of construction, maintenance, and/or rehabilitation (i.e. work zone) activities that restrict the capacity of the facility. The work zone operations category, on the other hand, reflects highway user costs associated with use of a facility during periods of construction, maintenance, and/or rehabilitation activities that generally restrict the capacity of the facility and disrupt normal traffic flow.

During normal operating conditions, as a general rule, there should be little difference between crash costs and delay costs as a result of pavement design decisions. Further, as long as the pavement performance levels remain relatively high and performance curves of the alternative designs are similar, there should be little, if any, difference between vehicle operating costs.

However, if pavement performance curves and levels differ substantially, significant vehicle operating cost differentials can develop. Figure 2. 2 depicts an exaggerated example of alternative pavement design strategies.

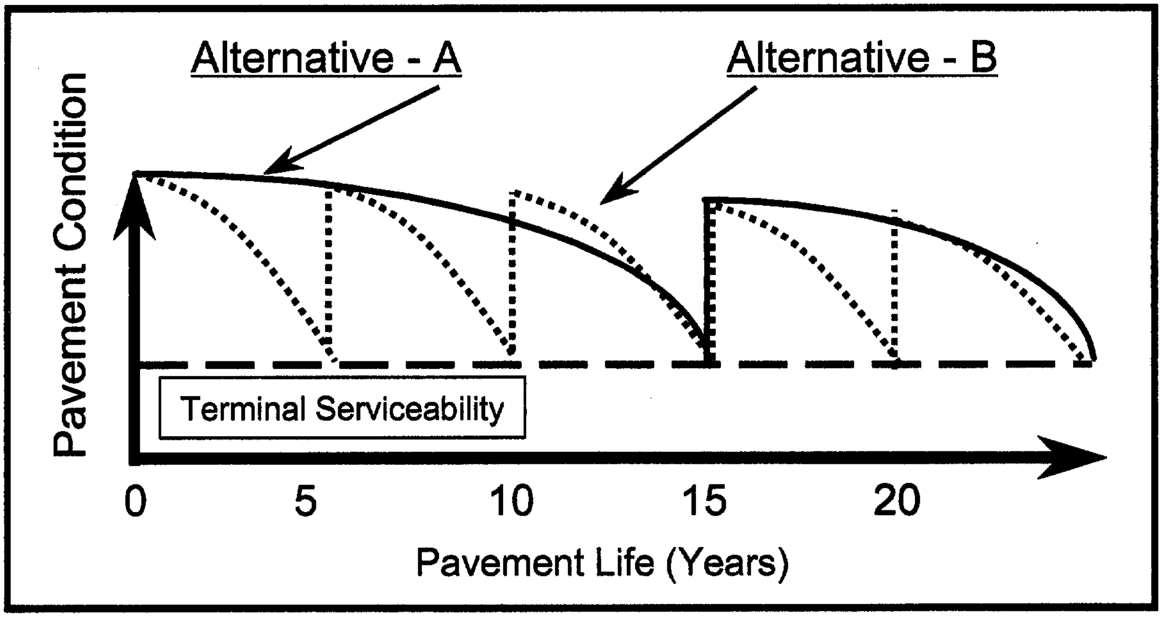

Figure 2. 2 Performance Curve verses Rehabilitation Strategy.

In Figure 2. 2 Alternative A represents a traditional longer-term strategy with rehabilitation implemented on a 15-year cycle. Alternative B consists of a minimal treatment on a 5-year cycle. This figure graphically depicts differences in performance levels for different rehabilitation strategies. Intuitively, differences in pavement performance may produce differences in vehicle operating costs. Slight differences in VOC rates due to differences in pavement performance characteristics (primarily roughness) when multiplied by several years VMT could result in huge VOC differentials over the life of the design strategy. This is particularly true for pavement preservation strategies that exhibit poor performance over most of the analysis period as shown by Alternative B in Figure 2. 2.

However, to calculate these differences the analysis must be able to (1) accurately estimate the pavement performance differences over time (at lest yearly) and, (2) Quantify the difference in VOC rates for slight differences in pavement performance at relatively high performance levels.

Figure 2. 3 shows the effect of road roughness, as measured by the international roughness index (IRI), on road user costs in New Zealand.(5) As shown in this figure, additional operating costs (as compared to a smooth road baseline) begin to accrue around an IRI equal to 170 inches per mile. According to work done by Darter, an IRI level of 170 is approximately equal to a PRS rating of 2.5. On higher order systems in the United States such as the National Highway System, SHAs typical consider pavements with a PSR of 2.5 to have reached their terminal serviceability index and in need of rehabilitation.

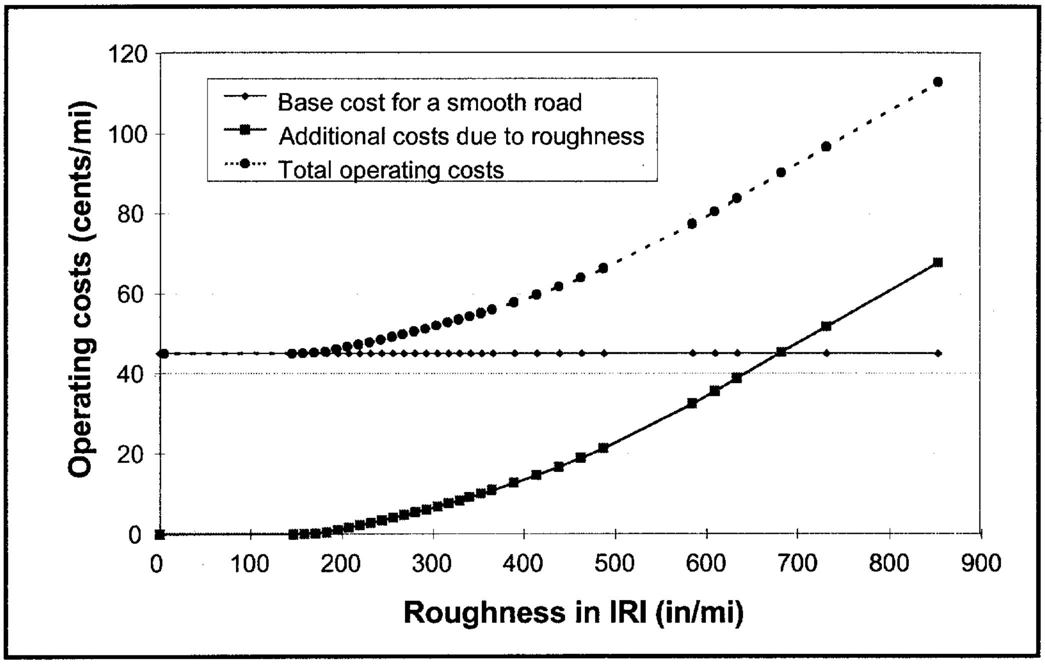

Figure 2. 3 Effect of Roughness on Road User Costs in New Zealand.

The effect of pavement condition on user operating cost at low roughness levels, if any, is not well documented at this time. There is, however, research currently underway in this area under NCHRP 1-33. NCHRP 1-33, "Methodology to Improve Pavement Investment Decision", has been initiated to obtain objective information relating costs associated with truck operating expenses (health claims, cargo damage, vehicle depreciation, maintenance and repair) with road roughness. This study is scheduled to be complete by 1999.

Additionally there is some preliminary work being done at Cornell University's School of Civil Engineering on establishing differential vehicle operating costs associated with pavement condition (i.e. IRI) for the New York State Department of Transportation: "Review and Development of Life-Cycle Cost and Network Analysis Procedures for NYSDOT Highway Pavements," Draft, August, 1997, Arnim Meyburg, John Mbwana, Neville Parker, and Patric Pranel.

Even if user operating cost differentials are established between smooth and very smooth roads, the analyst will still have to overcome the difficulty in estimating projected year by year performance differences between alternative pavement design strategies.

Considering the prevailing pavement performance ranges encountered in the United States on higher type facilities and the lack of precision in projecting year by year pavement performance differentials, this Interim Technical Bulletin does not advocate computation of user vehicle operating cost differentials during normal operating conditions.

On the other hand, during periods of initial construction and future maintenance and rehabilitation activities, (i.e. work zone operation), vehicle operating cost, user delay, and crash costs can be significantly different between alternative pavement design strategies. As a result, this technical bulletin focuses heavily on work zone user costs.

2.4.2 User Cost Rates

User cost rates as used here, refer to the dollar values assigned to each user cost component unit of measure. User costs are calculated by multiplying the quantity of the various additional user cost components (VOC, delay, and crash) incurred by the unit cost for those cost components.

Additional VOC are determined by multiplying the quantity of additional VOC incurred by the dollar value assigned to each VOC category (i.e. additional speed changes, stops, miles, hours of idling, etc). By the same token, user delay costs are determined by multiplying the additional hours of travel time due to WZ caused traffic delay (or additional miles of travel due to detours) by the dollar value of an hour of delay for each vehicle classification. Finally, the additional crash costs are determined by multiplying the number of additional crashes (by type) by the appropriate dollar value assign to each crash type.

Detailed procedures to calculate work zone user cost quantities of alternate pavement design strategies are presented in Chapter 3, while the unit costs associated with each cost component are discussed below.

VOC Rates

VOC rates for stopping, speed change, and idling cost values and delay times can be found in early work presented in table 5 of NCHRP report 133, *Procedures for Estimating Highway User Costs, Air Pollution, and Noise Effects.* This work is based on the earlier work of Robly Winfrey, *Economic analysis for Highways* (International ext Book Co., 1969). NCHRP 133 table 5 is reproduced as Table 2. 2.

Table 2. 2 Added Time and Vehicle Running Cost / 1000 Stops and Idling Costs (1970).

Initial Speed (mph)	Added Time (Hr/1000 Stops) (Excludes Idling Time)			Added Cost ($/1000 Stops) (Excludes Idling Time)		
	Pass Cars	Single Unit Truck	Combination Truck	Pass Cars	Single Unit Truck	Combination Truck
5	1.02	0.73	1.10	0.71	2.43	8.83
10	1.51	1.47	2.27	2.32	5.44	20.35
15	2.00	2.20	3.48	3.98	8.90	34.13
20	2.49	2.93	4.76	5.71	12.71	49.91
25	2.98	3.67	6.10	7.53	16.80	67.37
30	3.46	4.40	7.56	9.48	21.07	86.19
35	3.94	5.13	9.19	11.57	25.44	106.05
40	4.42	5.87	11.09	13.84	29.93	126.63
45	4.90	6.60	13.39	16.30	34.16	147.62
50	5.37	7.33	16.37	18.99	38.33	168.70
55	5.84	8.07	20.72	21.92	42.25	189.54
60	6.31	8.80	27.94	25.13	47.00	209.82
65	6.78	9.53	NA*	28.63	51.43	NA*
70	7.25	NA*	NA*	32.46	NA*	NA*
75	7.71	NA*	NA*	36.64	NA*	NA*
80	8.17	NA*	NA*	41.19	NA*	NA*
Idling Cost ($/Vehicle-Hour)				0.1819	0.2017	0.2166

Source: R. Winfrey, Economic Analysis for Highways (International Textbook Co., 1969) and Table 5 NCHRP Report 133

Added Cost ($/1000 Stops) - includes fuel, tires, engine oil, maintenance, and depreciation
Idling Cost ($/Veh-Hr) - includes fuel, engine oil, maintenance, and depreciation

*Original data did not provide values for trucks at higher speed. Analysts will need to extrapolate these values when truck calculations are needed at these higher speeds.

Table 2. 2 shows additional hours of delay and additional VOC associated with stopping 1000 vehicles from a particular speed and returning them to that speed. Different factors are provided for passenger cars, single unit and combination trucks. In addition, the table includes a vehicle operating cost associated with idling while stopped. The cost factors shown reflect 1970 prices based on a $3.00/hour value of time for passenger vehicles and $5.00/hour for all trucks.

To make these factors applicable to current day analysis, the values shown need to be escalated to reflect current/base year dollars. The escalation factor for VOC is determined by using the transportation component of the Consumer Price Index for the base year (1970) and the current year (Aug 1996). The transportation component of the consumer price index was 37.5 in 1970 and 142.8 in August of 1996. The VOC escalation factor used to escalate 1970 prices to August 1996 prices is:

$$\underset{\text{(VOC)}}{\text{Escalation Factor}} = \frac{142.8 \text{ (August 1996)}}{37.5 \text{ (1970)}} = 3.808$$

The added cost per 1000 stops columns in Table 2. 3 reflects the adjusted values using the above 3.808 index to establish new August 1996 base year prices.

Table 2. 3 Added Time and Vehicle Running Cost / 1000 Stops and Idling Costs (Aug 96).

Initial Speed (mph)	Added Time (Hr / 1000 Stops) (Excludes Idling Time)			Added Cost ($/1000 Stops) (Excludes Idling Time)		
	Pass Cars	Single Unit Truck	Combination Truck	Pass Cars	Single Unit Truck	Combination Truck
5	1.02	0.73	1.10	2.70	9.25	33.62
10	1.51	1.47	2.27	8.83	20.72	77.49
15	2.00	2.20	3.48	15.16	33.89	129.97
20	2.49	2.93	4.76	21.74	48.40	190.06
25	2.98	3.67	6.10	28.67	63.97	256.54
30	3.46	4.40	7.56	36.10	80.23	328.21
35	3.94	5.13	9.19	44.06	96.88	403.84
40	4.42	5.87	11.09	52.70	113.97	482.21
45	4.90	6.60	13.39	62.07	130.08	562.14
50	5.37	7.33	16.37	72.31	145.96	642.41
55	5.84	8.07	20.72	83.47	160.89	721.77
60	6.31	8.80	27.94	95.70	178.98	798.99
65	6.78	9.53	NA*	109.02	195.84	NA*
70	7.25	NA*	NA*	123.61	NA*	NA*
75	7.71	NA*	NA*	139.53	NA*	NA*
80	8.17	NA*	NA*	156.85	NA*	NA*
Idling Cost ($ / vehicle-hr)				0.6927	0.7681	0.8248

*Original data did not provide values for trucks at higher speed. Analysts will need to extrapolate these values when truck calculations are needed at these higher speeds.

While this table is designed to determine stopping cost, it can also be used to determine speed change cost. Speed change costs are the additional cost (VOC and delay) of slowing from one speed to another and returning to the original speed. Speed change costs are calculated by subtracting the cost and time factors of stopping at one speed from the cost and time factors of stopping at another speed. For example, the speed change costs of going from 55 mph to 40 mph and back to 55 mph is shown in Table 2. 4.

Table 2. 4 Speed Change Computations.

Initial Speed (mph)	Added Time (Hr / 1000 Stops) (Excludes Idling Time)			Added Cost ($/1000 Stops) (Excludes Idling Time)		
	Pass Cars	Single Unit Truck	Combination Truck	Pass Cars	Single Unit Truck	Combination Truck
55	5.84	8.07	20.72	83.47	160.89	721.77
40	4.42	5.87	11.09	52.70	113.97	482.21
55-40-55	1.42	2.20	9.63	30.77	46.92	239.56

Mileage Rates

In addition to these incremental vehicle operating costs associated with changes from normal operating condition, there is also a fundamental overall baseline VOC mileage rate associated with normal operating conditions. Typically this is expressed as an overall cents per mile rate. These rates would typically apply to any additional miles that must be driven due to detours.

Some readily apparent values are the marginal cost rates used by the Federal government. Federal travel regulations authorize the payment of $0.31/per mile for the use of privately owned passenger vehicles when used for official government travel. The flat mileage rate allowed by the IRS for business use of a privately owned passenger vehicle is also $0.31/mile (tax year 1996).

2.4.3 **Delay Cost Rates (Value of Time)**

Of all of the user costs rates, the cost rate assigned to user delay (i.e. the value of time) is by far the most controversial. As a result, the cost rates for user delay are discussed from several perspectives. The sources used in this Interim Technical Bulletin include updated values from earlier NCHRP research, recent guidance provided by the Office of the Secretary of Transportation (OST) and recently updated values used by FHWA in it's Highway Economic Requirements System (HERS) Model.

Earlier Research Studies

A base case value can be generated from the earlier NCHRP 133 report which used 1970 dollar values of $3.00/hour passenger vehicles and $5.00/hour for all trucks. Once again, these values must be escalated must be escalated to reflect current/base year values.

In this case, the escalation factor for the dollar value or time is determined by using changes to the "All Items Component" of the consumer price index for the base year (1970) and the current Year (Aug 1996). The "All Items Component" of the consumer price index was 38.8 in 1970 and 152.4 in August of 1996. The value of time escalation factor to escalate 1970 prices to August 1996 prices is:

$$\begin{array}{l} \text{Escalation Factor} \\ \text{(Value of Time)} \end{array} = \frac{152.4 \text{ (August 1996)}}{38.8 \text{ (1970)}} = 3.928$$

The updated values of time are shown in Table 2. 5

Table 2. 5 Up Dated 1970 Values of Time NCHRP 133 ($ / Delay-Hr.) (August 96).

Value Of Time	Pass Cars	Trucks	
		Single Unit	Combination
Value 1970	$3.000	$5.000	$5.000
Factor 8/96	3.928	3.928	3.928
Value 8/96	$11.78	$19.64	$19.64

More recent work (1993), from the computerized MicroBENCOST program developed under NCHRP 7-12, uses 1990 base year default values as shown in Table 2. 6. Table 2. 6 includes another escalation factor to bring these 1990 base year costs to a new, August 1996, base year. Once again, the escalation factor for the value of time is calculated by dividing the August1996 overall Consumer Price Index (152.4) by the overall Consumer Price Index for 1990 (130.7).

$$\begin{array}{c} \text{Escalation Factor} \\ \text{(8/96)} \end{array} = \frac{152.4 \text{ (August 1996)}}{130.7 \text{ (1990)}} = 1.166$$

Table 2. 6 Up Dated 1990 MicroBENCOST Default Values of Time (August 96).

Value of Time	Pass Cars	Single Unit Trucks		Combination Trucks	
		2 AX 12 Kips	3 AX 35 Kips	2S2 40 Kips	3S2 63 Kips
Value 1990	$9.75	$13.64	$16.28	$20.30	$22.53
		$14.96		$21.42	
Factor 8/96	1.166	1.166		1.166	
Value 8/96	$11.37	$17.44		$24.98	

Table 2. 7 includes both earlier NCHRP 133 and MicroBENCOST values and average values for the three vehicle classifications in August 1996 dollars per vehicle hour. Prior to using these values the analyst should escalate their value to reflect current costs using a similar approach.

Table 2. 7 Composite Earlier Research Values of Time (August 1996 $).

Updated Source	Pass Cars	Trucks	
		Single Unit	Combination
NCHRP 133	$11.78	$19.64	$19.64
MicroBENCOST	$11.37	$17.44	$24.98
Average Value	$11.58	$18.54	$22.31

OST Approach

Another source that can be used to determine the value of time is recent guidance provided by OST to the various DOT modal administrations. OST recommends using a percentage of the national wage rate for the value of time. OST recommended procedures apply different percentages of the national wage rate as a function of vehicle classification and trip type and purpose. Table 2. 8 provides ranges of the percentage of the national wage rate that should be applied to various combinations of trip type and purpose. As can be seen in Table 2. 8, business and truck travel are valued more highly than personal travel and inter-city personal travel is valued more highly than local personal travel.

Table 2. 8 Travel Time Ranges as a % of National Wage Rate: (Dollars/Person Hour).

Travel Category	Trip Type	
	Local	Inter-City
Personal	35 – 60 %	60 – 90 %
Business	80 – 120 %	80 – 120 %
Truck Drivers	100 %	100%

US DOT " The Value of Travel Time: Departmental Guidance for Conducting Economic Evaluations"

Table 2. 9 provides information on national hourly earning rates in 1995 Dollars.

Table 2. 9 Recommended Hourly Earning Rates (1995 Dollars per person hour).

Travel Category	Trip Type	
	Local	Inter-City
Personal	$17.00	$17.00
Business	$18.80	$18.80
Truck Drivers	$16.50	$16.50

US DOT " The Value of Travel Time: Departmental Guidance for Conducting Economic Evaluations"

Based on the information on appropriate percentages provided in Table 2. 8 and the recommended national hourly wage rate provided in Table 2. 9, OST developed the recommended ranges for the value of travel time shown in Table 2.10. The values associated with "Mixed" are the ranges to be used when the distribution between auto business and personal trips is not known. Hourly values shown for trucks are $16.50 for both local and inter city trip types.

Table 2.10 Ranges for Hourly Values of Travel Time (1995 U.S. $ per person-hour).

Travel Category	Local		Inter City	
	Low	High	Low	High
Personal	$ 6.00	$10.20	$10.20	$15.30
Business	$15.00	$22.60	$15.00	$22.60
Mixed	$ 6.40	$10.70	$10.40	$15.70
Truck Drivers	$16.50	$16.50	$16.50	$16.50

US DOT " The Value of Travel Time: Departmental Guidance For Conducting Economic Evaluations"

The ranges of values listed in Table 2.10 are dollars per person hour. As most user delay analyses compute vehicle hours of delay, these values must be adjusted for vehicle occupancy rates. According to the 1990 *Personal Transportation Survey*, typical auto vehicle occupancy rates for personal travel are 1.7 in urban areas and 2.0 in rural areas. The typical vehicle occupancy rates for trucks and auto business use is much closer to 1.0.

FHWA Highway Economic Requirements System (HERS) Model

The FHWA uses the HERS model in conducting national level analysis of highway performance, needs and economic evaluation of proposed highway improvements. Part of the economic analysis includes determining the value of travel time delay. The default dollar values of travel time used in the HERS model are shown in Table 2.11

Table 2.11 Value of One Vehicle Hour of Travel Time (1995 Dollars).

Travel Category	Autos		Trucks				
	Small	Medium	Single Unit			Combinations	
			4 Tire	6 Tire	3-4 Axle	4 Axle	5 Axle
Business	$27.99	$28.29	$20.20	$24.96	$27.02	$31.02	$31.58
Personal	$12.78	$12.78	$12.78	NA	NA	NA	NA
% AADT Personal Use	90%	90%	69%	0%	0%	0%	0%
Weighted Average	$14.30	$14.33	$15.08	$24.96	$27.02	$31.02	$31.58

FHWA Highway Economic Requirements System Technical Report v3-1 9/97 (Exhibit 8-7)

The values of travel time shown on lines one and two of Table 2.11 are per vehicle hour based on typical vehicle occupancy rates. The values associated with "% AADT Personal" on line three represents the percent of travel that is personal. The weighed average values shown on line four are for mixed flow of business and personal travel and are used when traffic flow distribution by travel category is not known. The values shown for trucks are significantly higher than either early research studies or the OST guidance. Part of the higher cost (approximately 30%) is attributable to value associated with the vehicle cargo and the vehicle itself.

Finally, another source that can be used to estimate the dollar values of time include tolls on toll roads in relation to time saved. One of the more interesting revelations of users' willingness to pay comes from the treatment of some High Occupancy Vehicle (HOV) facilities in California and Houston. Both States are experimenting with High Occupancy Toll (HOT) facilities... allowing Single Occupancy Vehicles (SOV) to use an HOV facility for a fee. These studies deal primarily with local auto personal trips and to some extent may reflect market prices. While the values inferred from such studies tend to indicate lower values of time (approximately $6.00/person hour), they may in reality reflect inefficient pricing policy rather than lower values.

Recommended Values of Travel Time (Dollars per Vehicle Hour)

Table 2.12 below is a composite table that brings the several sources of the value of time together.

Table 2.12 Composite Listing of Travel Time Values.

Source	Units	Autos	Trucks	Combination
US DOT – OST *	$ / Person Hr.	$10.80	$16.50	$16.50
MicroBENCOST	$ / Vehicle Hr.	$11.37	$17.44	$24.98
NCHRP	$ / Vehicle Hr.	$11.78	$19.64	$19.64
HERS	$ / Vehicle Hr.	$14.30	$25.99	$31.30

* Values for US DOT – OST reflect dollars per person hour

Based on consideration of all the potential sources, the ranges of the value of travel time per vehicle reflected in Table 2.13 are recommended for use in typical analyses where distribution data on trip purpose and type are not known.

Table 2.13 Recommended Values of Travel Time $/Vehicle Hour (Aug 1996).

Passenger Cars	Trucks	
	Single Unit	Combinations
$ 10 - $ 13	$17 - $19	$ 21 - $ 24

2.4.4 Crash Cost Rates:

Default crash cost rates are included in the MicroBENCOST software package developed for NCHRP research project 7-12. The default crash cost rates by crash type for both rural and urban settings are shown in Table 2.14

Table 2.14 MicroBENCOST Default Crash Cost Rates ($1,000) (1990 Dollars).

Intersection or Facility Type	Fatality		Non-Fatal Injury		Property Damage Only	
	Rural	Urban	Rural	Urban	Rural	Urban
RR Grade Crossing	$1,008	$994	$25.2	$13.3	$1.59	$3.09
Intersection / Interchange	$1,059	$932	$21.9	$14.3	$1.98	$1.35
Bridge	$1,111	$978	$24.9	$14.3	$2.14	$1.27
Highway Segment	$1,111	$978	$24.9	$14.3	$2.14	$1.27

The MicroBENCOST 1990 dollar default values shown in Table 2.14 are escalated to August 1996 dollars in Table 2.15 using the 1.116 escalation factor developed earlier.

Table 2.15 MicroBENCOST Default Crash Cost Rates ($1,000) (August, 1996 Dollars).

Intersection or Facility Type	Fatality		Injury		PDO	
	Rural	Urban	Rural	Urban	Rural	Urban
RR Grade Crossing	$1,125	$1,109	$28.1	$14.8	$1.77	$3.45
Intersection / Interchange	$1,182	$1,040	$24.4	$16.0	$2.21	$1.51
Bridge	$1,240	$1,091	$27.8	$16.0	$2.39	$1.42
Highway Segment	$1,240	$1,091	$27.8	$16.0	$2.39	$1.42

The MicroBENCOST default cost values for a fatality range from $1,091,000 to $1,182,000 which is quite a bit lower than the $2.700,000 cost per fatality averted recommended by the US DOT in a March, 14, 1995 memorandum from the assistant Secretary for Transportation Policy to DOT Modal Administrators. This $2,700,000 value is an update of the value originally recommended in basic guidance distributed on January 8, 1993.

In addition to the traditional direct work zone user costs, there are indirect user costs such as the impact of user delay on delivery fleet size, the costs associated with a rolling inventory, and other, indirect, impacts of delay to manufacturing plants which are now dependent on "*just-in-time*" delivery. While such factors will become more important over time, they are beyond the scope of these guidelines. It is interesting to note that cost related to delivery fleet size and the costs associated with a rolling inventory are included in the dollar value of delay time rates used in the FHWA HERS model.

2.5 DEVELOP EXPENDITURE STREAM DIAGRAMS

Expenditure stream diagrams are graphical representations of expenditures over time. They are generally developed for each pavement design strategy to help visualize the extent and timing of expenditures. A typical expenditure stream is shown in Figure 2. 4.

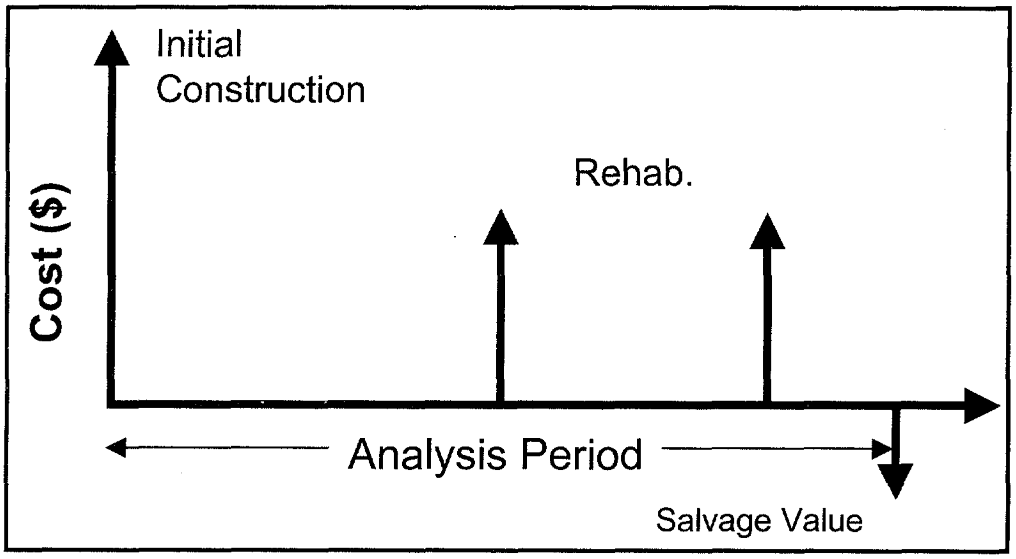

Figure 2. 4 Typical Expenditure Stream Diagram for a Pavement Design Alternative.

Normally costs are depicted as upward arrows at the appropriate time they occur during the analysis period and benefits are represented as negative cost or downward arrows.

In LCC analysis of pavement design alternatives, the basic benefits of providing and maintaining some pre-established pavement condition level on any given roadway are outside the scope of the analysis. The benefits of providing the specified level of pavement condition are considered to be the same for all pavement design strategies. As a result, the only benefits of concern are the differential costs associated between alternatives. The only benefit (i.e. the only downward arrow) would be the negative cost associated with any salvage value.

Under these conditions, the LCCA objective becomes finding the alternative pavement design strategy that meets the performance requirements at the lowest life cycle cost.

2.6 COMPUTE NET PRESENT VALUE

In its broadest sense, Life Cycle Cost Analysis (LCCA) is a form of economic analysis used to evaluate the long-term economic efficiency between alternative investment options. Economic analysis focuses on the relationship between costs, timings of costs, and discount rates employed. Once all costs and their timing have been developed, future costs must be discounted to the base year and added to the initial cost to determine the Net Present Value (NPV) for the LCCA alternative. As noted earlier, NPV is the economic indicator of choice and the basic NPV formula for discounting discrete future amounts at various points back to some base year in time is shown as follows:

$$NPV = \text{Initial Cost} + \sum_{k=1}^{n} \text{Rehab Cost}_k \left[\frac{1}{(1+i)^{n_k}} \right]$$

where: i = discount rate and
n = number of years into future

The $\left[\frac{1}{(1+i)^{n_k}} \right]$ component of the above formula is referred to as the Present Value (PV) factor for a single future amount. PV factors for various combinations of discount rates and future years are available in Discount factor tables, (more commonly referred to as interest rate tables). PV for a particular future amount is obtained by multiplying the future amount by the appropriate PV factor. Table 2.16 shows discount factors for a single future payment at 3, 4, and 5 percent discount rates for up to 40 years in the future.

Table 2.16 Present Worth Discount Factors: Single Future Payment.

	Discount Factor				Discount Factor		
Year	3%	4%	5%	Year	3%	4%	5%
1	0.9709	0.9615	0.9524	21	0.5375	0.4388	0.3589
2	0.9426	0.9246	0.9070	22	0.5219	0.4220	0.3418
3	0.9151	0.8890	0.8638	23	0.5067	0.4057	0.3256
4	0.8885	0.8548	0.8227	24	0.4919	0.3901	0.3101
5	0.8626	0.8219	0.7835	25	0.4776	0.3751	0.2953
6	0.8375	0.7903	0.7462	26	0.4637	0.3607	0.2812
7	0.8131	0.7599	0.7107	27	0.4502	0.3468	0.2678
8	0.7894	0.7307	0.6768	28	0.4371	0.3335	0.2551
9	0.7664	0.7026	0.6446	29	0.4243	0.3207	0.2429
10	0.7441	0.6756	0.6139	30	0.4120	0.3083	0.2314
11	0.7224	0.6496	0.5847	31	0.4000	0.2965	0.2204
12	0.7014	0.6246	0.5568	32	0.3883	0.2851	0.2099
13	0.6810	0.6006	0.5303	33	0.3770	0.2741	0.1999
14	0.6611	0.5775	0.5051	34	0.3660	0.2636	0.1904
15	0.6419	0.5553	0.4810	35	0.3554	0.2534	0.1813
16	0.6232	0.5339	0.4581	36	0.3450	0.2437	0.1727
17	0.6050	0.5134	0.4363	37	0.3350	0.2343	0.1644
18	0.5874	0.4936	0.4155	38	0.3252	0.2253	0.1566
19	0.5703	0.4746	0.3957	39	0.3158	0.2166	0.1491
20	0.5537	0.4564	0.3769	40	0.3066	0.2083	0.1420

NPV Computations:

Example NPV computations are provided for the following hypothetical problem. The example is based on a 35-year analysis period.

The initial pavement design will cost $1.1 million and have an associated work zone user cost of $300,000 at year 0. Additional rehabilitation cost of $325,000 will be incurred in years 15 and 30. Associated work zone user costs in years 15 and 30 will be $269,000 and $361,000 respectively. The salvage value at year 35, based on a prorated cost of the most recent 15-year rehabilitation design and remaining life, will be $108,300 (5/15 of $325,000).

The expenditure stream diagram for the example problem is shown in Figure 2. 5.

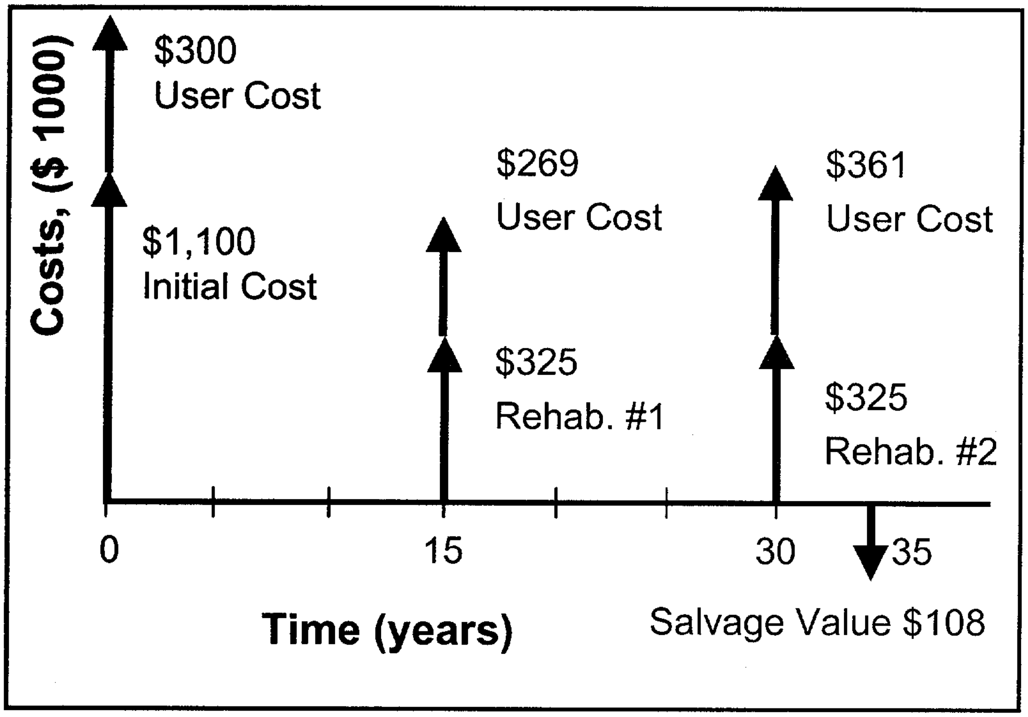
Costs, ($ 1000)
$300
User Cost
$1,100
Initial Cost
$269
User Cost
$325
Rehab. #1
$361
User Cost
$325
Rehab. #2
0
15
30
35
Time (years)
Salvage Value $108

Figure 2. 5 Expenditure Stream Diagram for Agency and User Costs.

Note that estimated user costs drop in year 15 and go back up in year 30. This is consistent with a longer duration initial work zone followed by short duration rehabilitation work zones impacted by continually increasing traffic volumes over time. Table 2.17 shows the results of PV computations using 4% PV factors for single future amounts for the example expenditure stream diagram. The bottom Line of Table 2.3 shows the NPV of the aggregated individual Present Values.

Table 2.17 Net Present Value Calculation Using 4 Percent Discount Rate Factors.

Cost Component Activity	Years	Costs ($1000)	Discount Factor	Discounted Cost ($1,000)
Initial Construction	0	1,000.0	1.0000	1,000
Initial Work Zone User Cost	0	300.0	1.0000	300
Rehab # 1	15	325.0	0.5553	180
Rehab # 1 Work Zone User Cost	15	269.0	0.5553	149
Rehab # 2	30	325.0	0.3083	100
Rehab # 2 Work Zone User Cost	30	361.0	0.3083	111
Salvage Value	35	-108.3	0.2534	-27
Total NPV				1913

2.7 ANALYZE RESULTS

Once completed, all LCCA should, at a minimum, be subjected to a sensitivity analysis. Sensitivity analysis is a technique used to determine the influence of major LCAA input assumptions, projections and estimates on LCCA results. In a sensitivity analysis, major input values are varied (either within some percentage of the initial value or over a range of values) while all other input values remain constant and the amount of change in results is noted. The input variables may then be ranked according to their effect on results. Sensitivity analysis allows the analyst to subjectively "get a feel for" the impact of the variability of individual inputs on overall LCCA results.

Many times a sensitivity analysis will focus on best case/worst case scenarios in an attempt to bracket outcomes. Most LCC sensitivity analysis, as a minimum, evaluate the influence of the discount rate used on LCCA results.

Sensitivity analyses may be carried out using common spreadsheet based applications such as Microsoft Excel, Lotus, or Quattro Pro. Table 2.18 and Table 2.19 below present the results of a spreadsheet analysis of the sensitivity of NPV of two example pavement design strategies to discount rate ranges from 2 to 6% for a 35-year analysis period. Total NPV at discount rates ranging from 2% to 6.0% are shown at the bottom of columns (e) - (i).

Table 2.18 Sensitivity Analysis – Alternative # 1.

Activity (a)	**Year** (b)	**Cost** ©	**NPV**				
			2.0% (e)	**3.0%** (f)	**4.0%** (g)	**5.0%** (h)	**6.0%** (i)
Construction	0	975	975	975	975	975	975
User Cost	0	200	200	200	200	200	200
Rehab #1	10	200	164	149	135	123	112
User Cost # 1	10	269	220	200	182	165	150
Rehab # 2	20	200	135	111	91	75	62
User Cost # 2	20	361	243	200	165	136	113
Rehab # 3	30	200	110	82	62	46	35
User Cost # 3	30	485	268	200	150	1,122	85
Salvage	35	-100	-50	-36	-25	-18	-13
Total NPV			2,266	2,081	1,934	1,815	1,718

Table 2.19 Sensitivity Analysis – Alternative # 2.

Activity (a)	**Year** (b)	**Cost** ©	**NPV**				
			2.0% (e)	**3.0%** (f)	**4.0%** (g)	**5.0%** (h)	**6.0%** (i)
Construction	0	1100	1,100	1,100	1,100	1,100	1,100
User Cost	0	300	300	300	300	300	300
Rehab #1	15	325	241	209	180	156	136
User Cost # 1	15	269	200	173	149	129	112
Rehab # 2	30	325	179	134	100	75	57
User Cost # 2	30	361	199	149	111	84	63
Salvage	35	-108	-54	-38	-27	-20	-14
Total NPV			2,166	2,025	1,914	1,825	1,753

Table 2.18 and Table 2.19 represent spreadsheet sensitivity analysis of the two pavement design strategies as a function of discount rate. It shows the distinctions between alternatives at different discount rates. Alternative #1 has a lower Initial agency cost, and, due to a shorter construction period, a lower user cost than Alternative 2. However, Alternative 1 requires 3 identical 10-year design rehabilitations compared to 2 identical 15-year design rehabilitations for alternative 2.

Out year user costs for alternative 1 increase as a result of increased traffic levels over time, while out year user costs for alternative 2 first decrease due to a shorter work zone period and then increase as a result of increased traffic levels over time.

Both alternatives have a remaining service life at year 35; Alternative #1 has 5 years and Alternative #2 has 10 years. The salvage value, as a prorated share of the last rehabilitation, for Alternative #1 is 50% (5 years remaining on a 10-year design) of its last rehabilitation cost. For Alternative #1, this translates into 50% of the $200,000 year 30 rehabilitation cost. The salvage value of alternative 2, on the other hand, is 66.6% (10 years remaining on a 15-year design) of it's last rehabilitation cost. This translates into 66.6% of the $325,000 year 30 rehabilitation cost.

Table 2.20 shows a direct comparison of the NPV of both alternatives at several discount rates. Inspection of Table 2.20 reveals that the NPV of both alternatives decrease as the discount rate increases. This results from the reduced present value of future costs at higher discount rates.

Because the amount and timing of future costs differ between alternatives, the effect on NPV is different. In this example alternative #1 is more expensive than alternative #2 at discount rates of 4% and lower, while alternative #2 is more expensive than alternative #1 at discount rates over 4%. These results are shown graphically in Figure 2. 6.

Table 2.20 Comparison of Alternative NPV's ($1000) to Discount Rate.

Activity	Discount Rate (%)				
	2.0	**3.0**	**4.0**	**5.0**	**6.0**
Total NPV Alternative # 1	2,266	2,081	1,934	1,815	1,718
Total NPV Alternative # 2	2,166	2,025	1,914	1,825	1,753
Cost Advantage Alt # 2 vs # 1	100	56	20	-10	-35

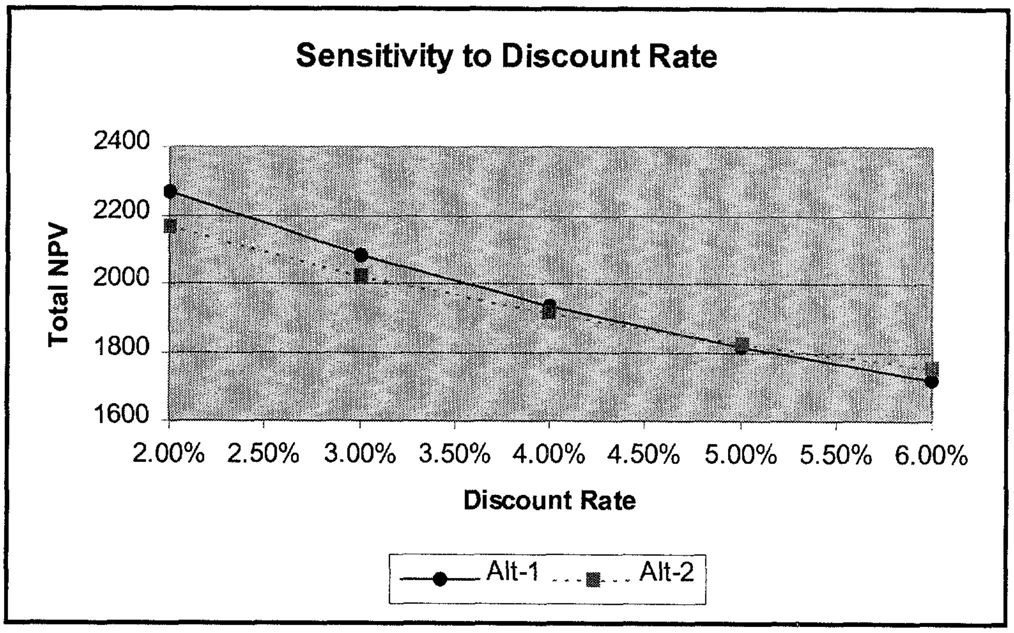

Figure 2. 6 Sensitivity of NPV for Alternatives 1 and 2 to Discount Rate.

Table 2.21 separates agency and user cost differences for the same range of discount rates.

Table 2.21 Sensitivity to User Cost and Discount Rate.

Cost Component	**Discount Rate, %**				
	2.0	**3.0**	**4.0**	**5.0**	**6.0**
Alternative # 1 Agency Cost	1,334	1,281	1,238	1,201	1,171
Alternative # 2 Agency Cost	1,467	1,404	1,353	1,312	1,278
Agency Cost Advantage Alt #2 vs # 1	-133	-123	-115	-111	-107
Alternative # 1 User Cost	932	800	696	613	547
Alternative # 2 User Cost	699	621	561	513	475
User Cost Advantage Alt # 2 vs # 1	233	179	135	100	72
Incremental Benefit / Cost (# 2 vs # 1)	1.75	1.46	1.17	0.91	0.67

Inspection of the Table 2.21 reveals that Alternative #2 has a higher agency cost than Alternative #1 at all discount rates considered. Further Alternative #2 has lower user cost than Alternative #1 at all discount rates considered.

The decision to include or not include user costs can significantly effect the LCCA results. In an effort to put the agency and user costs in perspective, the bottom row of Table 2.21 includes an incremental Benefit/Cost comparison of the reduction in user costs as a function of increased agency costs. The incremental Benefit/Cost data in Table 2.21 is computed by dividing the reduction in user costs (i.e. benefits) associated with selecting Alternative #2 in lieu of Alternative #1 by the added agency cost (costs) associated with selection of Alternative #2.

Similar sensitivity analyses could be conducted using other input variables such as agency cost, user costs, pavement performance lives, hourly dollar value of user delay, etc.

In addition to conducting a sensitivity analysis the analyst should examine the implications of contractor work hours on queuing costs as well as the anticipated maximum queue lengths and delay times. Queue lengths and associated user costs are discussed in detail in Chapter 3.

A primary drawback of the sensitivity analysis is that the analysis gives equal weight to any input value assumptions, regardless of the likelihood of occurring. In other words, the extreme values (best case and worst case) are given the same likelihood of occurrence as the expected value - which is not realistic. A powerful analytical technique to overcome this limitation is presented in Chapter 4.

2.8 RE-EVALUATE DESIGN STRATEGY

Once the net present values have been computed for each alternative and limited sensitivity analysis performed, the analyst needs to step back and reevaluate the competing design strategies. As noted in Chapter1, the overall benefit of conducting a life cycle cost analysis is not necessarily the LCCA results themselves, but rather how the information resulting from the analysis can be used by the designer to modify the proposed alternatives and develop more cost effective strategies.

For example, if user costs dwarf agency costs for all the alternatives, the analysis may indicate that none of the alternatives analyzed are viable. It could indicate that the designer needs to evaluate the current design strategies impacts on future maintenance of traffic…that the design strategies need to reflect the need for additional capacity in the out-years to mitigate the impact on highway users. The solution to out-year capacity problems could include enhanced structural design of the shoulders in early-year pavement designs to allow for the use of the shoulder in subsequent rehabilitation traffic control plans. It also could include enhanced structural design of the main-line pavement to minimize the frequency of subsequent rehabilitation efforts and "designing in" features that will make future rehabilitation go more smoothly. Other options available include: revising the maintenance of traffic plans, reducing the construction period, restricting the contractor's work hours or imposing "lane rental" fees, planning ahead for additional lanes/routes, and even examining programs to temporarily shift traffic to alternative modes of travel. It is important to note that restricting the contractor's hours of operation or the number of work days allowed will more than likely result in increase to agency cost.

LCCA results are not as important as the overall logical analytical process through which the results are obtained. Many factors influence the ultimate selection of a pavement design strategy. LCCA results are just one of many factors. The final decision may include a number of additional factors outside the LCCA process. Some of these may include local politics, availability of funding, industry capability to perform the required construction, agency experience with a particular pavement type, as well as the accuracy of the pavement design and rehabilitation models. Such other factors are discussed further in Chapter 3 of the 1993 *AASHTO Pavement Design Guide*. When such "other factors" weigh heavily in the final pavement design selection, their influence on the final decision should be documented.

Many assumptions, estimates, and projections feed the LCCA process. The variability associated with these inputs can have a major influence on the confidence the analyst can place in LCCA results. It all depends on the accuracy of the inputs used. The accuracy of LCCA results is directly dependent on the analyst's ability to accurately forecast future costs, pavement performance, traffic, etc over 30 years into the future. To effectively deal with the uncertainty associated with such forecasts, a probabilistic risk analysis approach (as presented in Chapter 4) becomes more and more essential to quantitatively capture the uncertainty associated with input parameters in LCCA results.

CHAPTER 3 - WORK ZONE USER COST

3.1 PURPOSE

This chapter presents a rational, step by step, procedure to allow the analyst, based on capacity flow analysis, to determine user costs associated with the establishment of a work zone. User cost rates were discussed earlier in chapter 2. This chapter focuses on calculating the quantity associated with each of the individual user cost components (VOC, User Delay, and Crashes) and the computation of overall user costs in a spreadsheet environment. Automated computer programs are discussed at the end of the chapter.

While there are microcomputer-based user cost analysis software programs currently available, it is important that the analyst have a thorough understanding of the principles involved before attempting to apply them. By understanding the major factors influencing work zone user costs, the analyst can take steps to minimize the effect of planned future rehabilitation activities on highway users.

3.2 WORK ZONE USER COSTS

Work zone user costs are the increased VOC, delay and crash costs to highway users resulting from the establishment of construction, maintenance, or rehabilitation work zones. They are a function of the timing, duration, frequency, scope, and characteristics of the work zone; the volume and operating characteristics of the traffic affected; and the dollar cost rates assigned to vehicle operating, delay, and crashes. Each of these issues will be addressed in the sections that follow.

In the end, user cost are computed by multiplying the quantity of additional VOC, delay, and number of crashes by the unit cost rates assigned to these components.

3.3 WORK ZONE DEFINED

Work Zone is defined in the Highway Capacity Manual as an area of a highway in which maintenance and construction operations are taking place that impinge on the number of lanes available to traffic or effect the operational characteristics of traffic flowing through the area.

Each work zone established over the analysis period can have different impacts on traffic flow and associated user costs and must be evaluated as a separate event. Whenever characteristics of the work zone or the characteristics of the affected traffic change, a separate work zone must be defined and analyzed.

Pavement design performance differences directly effect the frequency and timing of maintenance and rehabilitation activities. Pavement rehabilitation and maintenance activities generally occur at different points in the analysis period with different traffic and generally vary in scope and duration. The point in time that they occur also affects the influence of the discount factor used in developing NPV.

3.4 WORK ZONE CHARACTERISTICS

In order to analyze work zone user costs, work zone characteristics associated with alternative designs and supporting maintenance and rehabilitation strategies must be defined as part of the development of alternative pavement design strategies. Alternative pavement preservation strategies must include how often (the number of times) the facility will be under construction, maintenance or rehabilitation activities and the year at which work zones are anticipated. Strategies should also include estimates of the number of days the work zone will last, the hours of the day the work zone will be in place, and the anticipated maintenance of traffic strategy.

Work zone characteristics of concern include such factors as work zone length, number and capacity of lanes open, duration of lane closures, timing (hours of the day and days of the week) of lane closures, posted speed, and the availability and physical and traffic characteristics of alternative routes. The strategy for the maintenance of traffic should include any anticipated restrictions on contractor's or maintenance force's hours of operations or ability to establish lane closures. Specific details in a LCCA should include:

- ☐ Projected year work zones occur (years 5, 8, 12, etc)
- ☐ Number of days the work zone will be in place (construction period)
- ☐ Specific hours of each day, as well as, the days of the week the work zone will be in place.
- ☐ Work zone length and posted speed

The duration of a work zone (i.e. the overall length of time a facility or portion of a facility is out of service) can range from sporadic daily lane closures for maintenance to several months for bridge deck replacements).

As noted earlier, the differential routine maintenance cost between alternative pavement design strategies tends to be insignificant when compared to initial construction and rehabilitation costs. To a large extent, the same is true of user costs resulting from routine reactive type maintenance activities. Routine maintenance work zones tend to be relatively infrequent, of short duration, and outside of periods of peak traffic flow. As such, analysts should focus their attention on user costs associated with major work zones.

3.5 TRAFFIC CHARACTERISTICS

User costs are directly dependent on the volume and operating characteristics of the traffic on the facility. Each construction, maintenance, and rehabilitation activity generally involves some temporary impact on traffic using the facility. The impact can vary from insignificant for minor work zone restrictions on low volume facilities to highly significant for major lane closures on high volume facilities.

The major traffic characteristics of interest for each year a work zone will be established include: (1) the overall projected Average Annual Daily Traffic (AADT) volumes on both the facility and possibly alternate routes, (2) the associated 24-hour directional hourly demand distributions, and (3) the vehicle classification distribution of the projected traffic streams. On high volume routes distinctions between weekday and weekend traffic demand and hourly distributions becomes important. Further, when work zones are proposed on recreational routes during seasonal peak periods, seasonal AADT traffic distribution also becomes important.

3.5.1 AADT

Current AADT volumes are normally readily available for the base year and projected compound traffic growth rates can be obtained from the traffic monitoring section. From these two pieces of information calculation of future year work zone AADT is relatively simple.

While the calculations for future year AADT are rather straight forward, there are two major issues associated with the reasonableness of such projections on high volume urban facilities. The first is whether the roadway in question can handle large projected volume increases and the second is whether traffic using a facility will continue to use a facility when work zones are established and traffic flow is restricted.

In the first issue, design capacity and ultimate capacity are quite different. Design capacity is generally set to handle the design year 30th highest hour volume at level of service D or E. However, Table 2.1 of the 1994 Highway Capacity Manual shows measured maximum 24 hour traffic volumes for some of the more heavily traveled Interstate urban freeways which far exceed the traffic volumes normally associated with level of service D or E. Reasonableness dictates that AADT projections should not be allowed to exceed the maximum observed 24-hour traffic volumes contained in Table 2.1 of the HCM.

The second issue, related to whether traffic using a facility will continue to use a facility when work zones are established and traffic flow is restricted, is discussed under the Traffic Diversion section which follows.

3.5.2 Traffic Diversion

Traffic demand is generally determined based on facility operating characteristics during periods of normal operations. Traffic demand during work zone operations may or may not be the same. Some portion of the traffic normally wanting to use the facility may divert to other routes when work zones are established.

Vehicles use a given facility because it offers, what the vehicle operators perceive to be, the least expensive combination of vehicle operating and time delay costs, consistent with safety requirements. When faced with restricted flow, or even the anticipation of restricted flow, vehicle operators who normally use a facility will exercise one of several options. The several potential behavioral responses are somewhat whimsically categorized below.

Hang Toughers - This group continues to use the facility as they always have. They are primarily users with little, or no, option. They (1) must make the trip; (2) they must make it at a specific time, and (3) either don't know of, or don't have, alternative routes or modes to choose from. These users pay the full price of the work zone and have little effect on other facilities in the corridor. In rural areas the predominate choice of through traffic will be to tough it out, as these users generally must make the trip, and do not have much information on alternative routes unless formal detours are established.

Time Shifters - Time Shifters have the ability, and choose to, travel on the facility at a different time – generally a time well outside of the restricted flow period. These users lessen their impact by sharing the impact with other vehicles by "invading" their time slot. These users also have little effect on other facilities in the corridor, but do impact hourly traffic distribution.

Detourees - Detourees either, seek out and use alternate routes, or are forced to negotiate detours established by the highway agency. These operators also lessen their impact by sharing the impact with other vehicles by "invading" their routes. They tend to trade off anticipated time delay for additional travel distances and associated vehicle operating costs. In urban areas this could include users who switch modes. Detourees can have significant impact on over all user costs of alternative routes. Sections 3.8, Circuity, and 3.9, Crash Costs, address the additional user costs associated with the additional mileage traveled by detoured and diverted traffic.

Trip Swappers - Trip Swappers have the luxury of totally abandoning the trip or seeking other destinations when the cost, in terms of time and money, becomes too great. Historically, this group consists primarily of shopping and social/recreational trip makers. While their behavior may diminish the user cost impact of the work zone they adversely impact businesses along the route in question. More recent trends in people working out of the home and telecommuting may have a significant effect on work trips in the future.

Handling Diversion

In simple cases, where either work zone disruption is tolerable or alternative routes are limited, estimated AADT during the year of the work zone can be anticipated to continue on the facility and the work zone analysis can be limited to the existing facility.

In more complex situations where existing traffic would face intolerable work zone disruptions, it is entirely possible that total travel demand and hourly distribution on the facility may change when the work zone is established. When demand changes, the scope of work zone user cost analysis may have to expand beyond the existing facility and include user cost changes on major alternative routes. When preliminary analysis of travel demand show that work zone user delays are unreasonably high, the analyst should especially seek help from the traffic engineering section in generating revised traffic demand forecasts during work zone operations.

An alternative approach is to assume that those extraordinary user costs are truly there, and time and route shifters are merely diminishing their impact by sharing the costs with users of other routes and time slots. In short, the misery's still there – its just being shared differently.

3.5.3 Vehicle Classification

Highway user costs are a composite of the costs of all effected highway users. Highway users are not a homogeneous group. They include commercial and non-commercial vehicles ranging from passenger vehicles through the heaviest trucks. These different vehicle types have different operating characteristics and associated operating costs. Further, the value of user delay differs between vehicle classes. As a result, user costs need to be analyzed for each major vehicle class present in the traffic stream.

There are many truck vehicle classifications representing various size and weight configurations. Appendix A of the FHWA Traffic Monitoring Guide, Third Edition (February, 1995) includes 13 different vehicle classifications. User cost analysis based on 13 vehicle classes is much too detailed for the level of sophistication in the analysis procedures proposed.

A more reasonable approach is to use the following three broad vehicle classes:

1. Passenger cars and other 2 axle, 4-tired passenger vehicles (classification types 1-3)
2. Single unit trucks, two-axle, 4-tired or more commercial trucks (classification types 4-7)
3. Combination unit trucks (classification types 8-13)

On high traffic volume facilities, user delay cost are likely to represent a very significant component of overall user costs in a pavement design LCCA, particularly when vehicle demand exceeds capacity. As passenger vehicles represent the bulk of vehicles in the traffic stream, analysts may find it beneficial to subdivide passenger vehicles into commercial and non-commercial categories. Further, although vehicle occupancy rates have consistently fallen over time, they cannot generally be ignored.

3.5.4 Directional Hourly Traffic Distribution

The estimated hourly traffic distribution during work zone operations is essential to be able to compare the unrestricted demand on the facility with the facility's ability to carry that traffic through the work zone. Hourly distribution specific to a particular facility can be developed from agency traffic data or general distribution data developed for various functional class and area type combinations. On high volume routes, the difference between weekday and weekend traffic volumes and hourly distributions may need to be considered.

Table 3. 1 represents default directional hourly distributions for Urban and Rural Roadways included in MicroBENCOST.

Table 3. 1 includes hourly directional distributions factors associated with inbound and outbound directions. The hourly demand is computed by multiplying the AADT by the hourly percent and directional factors for the direction in question.

Hourly Volume = %ADT x Hourly Distribution Factor x Hourly Directional Factor

For example, the inbound directional hourly demand volume between 8 and 9 A.M. on an urban roadway with a 40,000 AADT would be:

40,000 AADT x 6.3% x 59% = 1487 vehicles/hour

Although MicroBENCOST provides for the use of different directional hourly distribution values for Interstate, Principal Arterial, Minor Arterial, and Major Collector, the default values listed are the same for all functional classes. Pennsylvania, on the other hand, does provide

Table 3. 1 Default Hourly Distributions from MicroBENCOST (All Functional Classes).

Hour	Rural			Urban		
	% ADT	Direction		% ADT	Direction	
		In	Out		In	Out
0 – 1	1.8	48	52	1.2	47	53
1 – 2	1.5	48	52	0.8	43	57
2 – 3	1.3	45	55	0.7	46	54
3 – 4	1.3	53	47	0.5	48	52
4 – 5	1.5	53	47	0.7	57	43
5 – 6	1.8	53	47	1.7	58	42
6 – 7	2.5	57	43	5.1	63	37
7 – 8	3.5	56	44	7.8	60	40
8 – 9	4.2	56	44	6.3	59	41
9 – 10	5.0	54	46	5.2	55	45
10 – 11	5.4	51	49	4.7	46	54
11 – 12	5.6	51	49	5.3	49	51
12 – 13	5.7	50	50	5.6	50	50
13 – 14	6.4	52	48	5.7	50	50
14 – 15	6.8	51	49	5.9	49	51
15 – 16	7.3	53	47	6.5	46	54
16 – 17	9.3	49	51	7.9	45	55
17 – 18	7.0	43	57	8.5	40	60
18 – 19	5.5	47	53	5.9	46	54
19 – 20	4.7	47	53	3.9	48	52
20 – 21	3.8	46	54	3.3	47	53
21 – 22	3.2	48	52	2.8	47	53
22 – 23	2.6	48	52	2.3	48	52
23 – 24	2.3	47	53	1.7	45	55

different hourly distributions for various functional class roadways, however, they do not provide a directional split factor. Hourly distribution factors used by PA-DOT in developing user delay cost are contained in Table 3. 2.

Table 3. 2 Pennsylvania DOT AADT Distribution (Hourly Percentages).

Hour	Traffic Pattern Group					
	Interstate		Principal Arterial		Minor Arterial	
	Urban	Rural	Urban	Rural	Urban	Rural
12 – 1	1.3	1.7	0.9	0.9	0.8	0.7
1 – 2	0.9	1.4	0.5	0.5	0.4	0.4
2 – 3	0.8	1.3	0.4	0.5	0.3	0.3
3 – 4	0.8	1.3	0.4	0.5	0.3	0.4
4 – 5	1.1	1.4	0.6	0.9	0.4	0.8
5 – 6	2.1	2.1	1.8	2.3	1.3	2.2
6 – 7	4.7	3.7	4.4	4.9	4.0	4.5
7 – 8	6.4	4.9	6.2	6.2	6.4	5.5
8 – 9	5.6	4.9	5.7	5.5	5.7	5.3
9 – 10	5.1	5.2	5.1	5.3	4.8	5.4
10 – 11	5.2	5.5	5.2	5.4	4.9	5.8
11 – 12	5.4	5.8	5.6	5.6	5.5	6.0
12 – 13	5.5	5.7	6.0	5.7	6.0	6.2
13 – 14	5.5	5.9	5.9	5.9	5.7	6.4
14 – 15	6.1	6.3	6.4	6.6	6.3	7.2
15 – 16	7.3	6.9	7.4	7.7	7.6	8.1
16 – 17	7.8	7.2	7.8	8.0	8.3	8.0
17 – 18	7.2	6.6	7.5	7.4	8.0	7.1
18 – 19	5.4	5.3	5.9	5.5	6.2	5.4
19 – 20	4.3	4.4	4.8	4.3	5.1	4.4
20 – 21	3.7	3.8	4.0	3.6	4.3	3.6
21 – 22	3.2	3.4	3.3	3.0	3.4	2.9
22 – 23	2.6	2.9	2.4	2.3	2.4	2.1
23 – 24	2.0	2.4	1.7	1.5	1.6	1.4

3.6 CONCEPTUAL ANALYSIS

Before addressing user cost calculation procedures, it is helpful to conduct a conceptual analysis of a work zone operation. There are a total of seven possible work zone user cost components that can occur; three associated with a "Base Case" situation were traffic operates under "Free Flow" conditions, and four associated with a "Queue" situation were traffic operates under "Forced Flow" conditions. Each of the potential user costs are discussed conceptually in the Free Flow and Forced Flow (level of service "F") sections that follow.

3.6.1 Free Flow

Work zones restrict traffic flow either by reducing capacity or, as a minimum, by posting lower speed limits. Figure 3. 1 shows free flow conditions at a work zone. All traffic that flows through the work zone, at a minimum, must slow down while traveling through it and then accelerate back to normal operating speed. This is commonly referred to as a speed change and results in three work zone related User Cost components; speed change delay, speed change VOC, and reduced speed delay. The cost components associated with the free flow case are graphically presented in Figure 3. 1 and are described below.

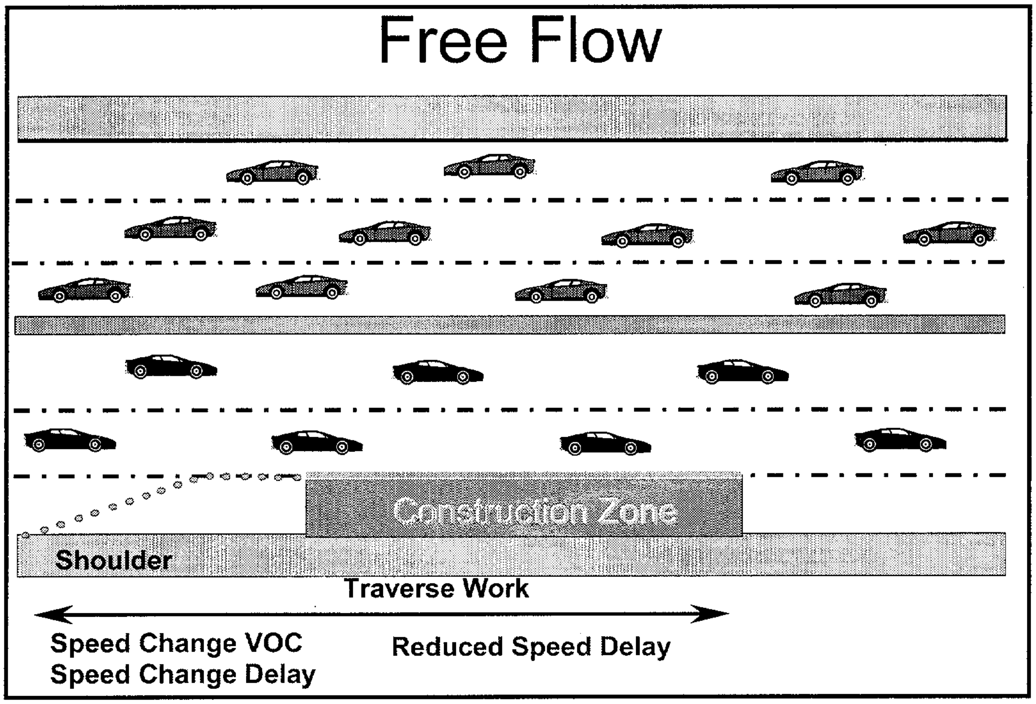
Free Flow
Construction Zone
Shoulder
Traverse Work
Speed Change VOC
Speed Change Delay
Reduced Speed Delay

Figure 3. 1 Free Flow Cost Components.

Speed Change Delay - This is additional time necessary to decelerate from the unrestricted upstream approach speed to the work zone speed and then to accelerate back to the initial approach speed after traversing the work zone.

Speed Change VOC - This is additional vehicle operating cost associated with decelerating from the unrestricted upstream approach speed to the work zone speed and then accelerating back to the approach speed from the work zone speed after traversing the work zone.

Reduced Speed Delay - This is additional time necessary to transverse the work zone at the lower posted speed and depends on the normal and work zone speed differential and length of the work zone.

If traffic demand remains below work zone capacity, added work zone user costs are limited to the above three components and the analysis is relatively simple. In most cases delay times remain relatively low and represent more of a minor irritation and inconvenience than a serious problem.

3.6.2 Forced Flow (Level of Service F)

When hourly traffic demand exceeds work zone capacity, traffic flow breaks down and a queue of vehicles develops as shown in Figure 3. 2. It is important to note that the queue does not form in the work zone itself, but in the upstream approach to the work zone.

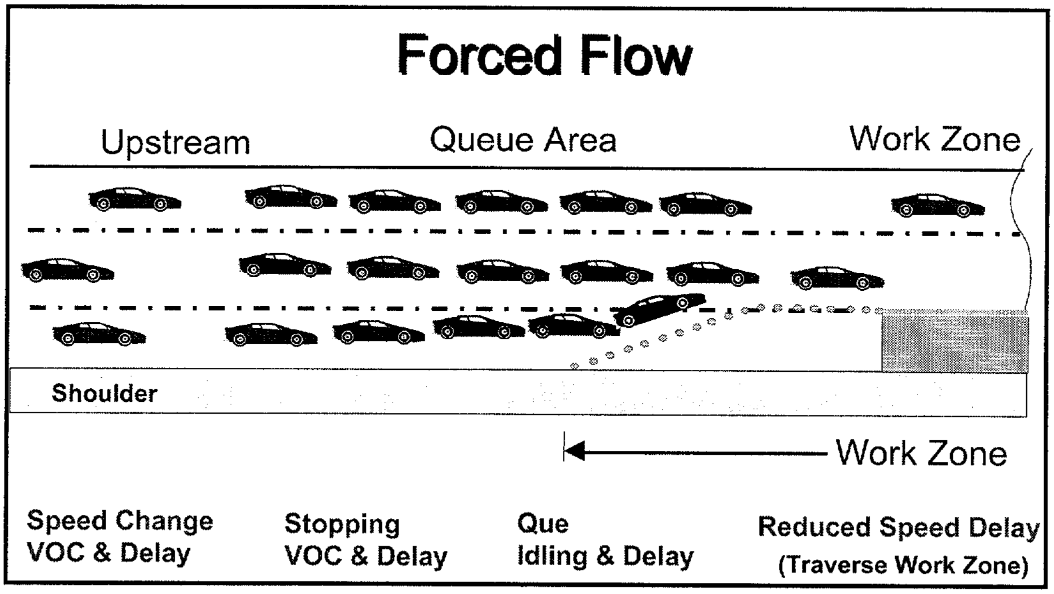
Forced Flow
Upstream
Queue Area
Work Zone
Shoulder
Work Zone
Speed Change VOC & Delay
Stopping VOC & Delay
Que Idling & Delay
Reduced Speed Delay (Traverse Work Zone)

Figure 3. 2 Forced Flow Level of Service "F".

Once a queue develops, all approaching vehicles must not only slow down before proceeding through the work zone itself, but must also stop at the approach to the work zone and creep through the length of the physical queue under forced flow conditions at significantly reduced speeds. As long as demand exceeds capacity, the length of the queue grows, exacerbating the problem. When demand eventually falls below capacity, or when capacity is increased above demand by removing the work zone restriction, vehicles then leave the queue faster than they arrive and the length of the queue shrinks and eventually dissipates over time. When capacity is reduced on high traffic facilities, it is not uncommon for queues to develop in the morning peak traffic period, dissipate, and then redevelop in the afternoon peak traffic period. In exceptionally congested areas, queues may form early in the morning and continue throughout the day and into the evening hours.

Queuing situations impose four more work zone related user costs that are a direct result of the queue. They are in addition to the "Free Flow" added user costs associated with the work zone itself. They only apply to vehicles that encounter a physical queue. They include:

Stopping Delay - This is additional time necessary to come to a complete stop (instead of just slowing to the work zone speed) and the additional time to accelerate back to the work zone speed.

Stopping VOC - This is additional vehicle operating cost associated with stopping and accelerating back up to work zone speed.

Queue Delay - This is additional time necessary to creep through the queue under forced flow conditions.

Idling VOC - This is the additional vehicle operating costs associated with "stop and go" driving in the queue. The idling cost rate multiplied by the additional time spent in the queue is an approximation of actual VOC associated with stop and go conditions.

The conceptual analysis presented here is geared primarily to freeway conditions. Conceptual analysis of other facilities with at grade intersections would also incur speed change, stopping, delay, and idling cost, but at a much higher frequency due to intersection control devices and turning movements.

3.7 COMPUTATIONAL ANALYSIS

Once the individual work zones have been identified, each is evaluated separately. This is the point at which individual user cost components are quantified and converted to dollar cost values. This section provides an approach for actually quantifying and costing the individual work zone user cost components encountered. The overall steps involved are listed below. Each step will be discussed in detail in the following sections.

1. Project Future Year Traffic
2. Calculate Work Zone Directional Hourly Demand
3. Determine Roadway Capacity
4. Identify the User Cost Components
5. Quantify Traffic Affected by Each Component
6. Compute Reduced Speed Delay
7. Select and Assign VOC Cost Rates
8. Select and Assign Delay Cost Rates
9. Assign Traffic to Vehicle Classes
10. Compute Individual User Costs Components by Vehicle Class
11. Sum Total Work Zone User Costs
12. Address Circuity and Crash Costs

Example Work Zone Problem Defined

The following information describes the work zone that will be used as "the example problem" throughout the remainder of this section to illustrate the work zone user cost computational steps described above.

Facility: 3-lane directional out-bound Interstate. Posted speed is 55 mph and grades on the facility are less than 2%.

Traffic: Base year (1995) AADT of 122,000 vehicles per day. Vehicle classification counts indicate the following traffic stream mix: 90% Passenger Vehicles, 5.4% Single Unit Trucks, and 4.6% Combination Trucks. The 10-year projected traffic growth rate for the various vehicle classifications are 2.2% for passenger vehicles and 5% per year for both single unit and combination trucks.

Work Zone: 5.25 mile long single lane closure from 9 AM until 3 PM and from 8 PM until 5 AM the following morning. Work zone posted speed is 40 mph. The work zone will be in place for 60 days in 1999.

3.7.1 **Project Future Year Traffic Demand**

The first step is to project future year hourly traffic demand volumes for each vehicle class for the year the work zones will be in place, from current or base year AADT, using compound traffic growth factors. The following formula applies.

$$\text{Future Year AADT} = \text{Base Year AADT} \times \text{Vehicle class \%} \times (1 + \text{growth rate})^{(\text{Future Yr.} - \text{Base Yr.})}$$

AADT on a facility in 1999 can be determine from a 1995 base year AADT of 122,000 using the above formula by applying the appropriate compound growth rate factor (in this example 2.2% for passenger vehicles and 5% for trucks) as follows:

Projected 1999 AADT

Passenger Vehicles	122,000 x 0.900 x $1.022^{(4)}$	=	109,800 x 1.09095	=	119,786	=	89%
Single Unit Trucks	122,000 x 0.054 x $1.050^{(4)}$	=	6,588 x 1.21551	=	8,008	=	6%
Combination Trucks	122,000 x 0.046 x $1.050^{(4)}$	=	5,612 x 1.21551	=	6,821	=	5%
Total Traffic					134,615	=	100%

Based on these new numbers total traffic in 1999 will be 134,615 and, due to the differential traffic growth rates for trucks, the 1999 vehicle mix will be approximately 89 % for passenger vehicles, 6% for single unit trucks, and 5% for combination trucks.

3.7.2 Calculate Work Zone Directional Hourly Demand

Directional hourly traffic distribution should be determined from agency traffic data on the roadway being analyzed or from traffic data on similar facilities. If however, such data is not available, the default hourly distributions for various roadway types in urban and rural settings from MicroBENCOST (reproduced in Table 3. 1) can be used. Table 3.3 below contains the future year directional hourly demand for the example problem generated using the default MicroBENCOST urban outbound hourly distribution factors and 134,615 future year AADT.

Inspection of table 3.3 reveals that AM outbound demand peaks at 4,200 vehicles per hour in the 7:00 to 8:00 am period, while the PM outbound demand peaks at 6,885 vehicles per hour in the 5:00 PM to 6:00 PM time period.

3.7.3 Determine Roadway Capacity

In analyzing work zone user costs, there are three capacities that need to be determined. They include: (1) the free flow capacity of the facility under normal operating condition, (2) the capacity of the facility when the work zone is in place, and (3), the capacity of the facility to dissipate traffic from a standing queue. Each of these is discussed in turn.

Table 3. 3 Work Zone Directional Hourly Demand (All Vehicle Classes).

AADT 134,615	Out Bound Urban Interstate		
Hour	% ADT	Directional Factor	Demand
12 – 1	1.8	53	856
1 – 2	1.5	57	614
2 – 3	1.3	54	509
3 – 4	1.3	52	350
4 – 5	1.5	43	405
5 – 6	1.8	42	961
6 – 7	2.5	37	2,540
7 – 8	3.5	40	4,200
8 – 9	4.2	41	3,477
9 –10	5.0	45	3,150
10 –11	5.4	54	3,417
11 –12	5.6	51	3,639
12 –13	5.7	50	3,769
13 –14	6.4	50	3,837
14 –15	6.8	51	4,051
15 –16	7.3	54	4,725
16 –17	9.3	55	5,849
17 –18	7.0	60	6,865
18 –19	5.5	54	4,289
19 –20	4.7	52	2,730
20 –21	3.8	53	2,354
21 –22	3.2	53	1,998
22 –23	2.6	52	1,610
23–24	2.3	55	1,259
Total			67,453

Free Flow Capacity – is the maximum capacity a facility can handle under free flow conditions. According to the 1994 HCM (page 2-10), the maximum capacity for a 2 lane directional freeway under ideal conditions is 2,200 passenger cars per hour per lane (pcphpl) and 2,300 pcphpl for a 3 or more lane directional freeway. The HCM points out that the above capacities under ideal conditions must be reduced for such real world factors as restricted lane widths, reduced lateral clearances, the presence of trucks and recreational vehicles, and the presence of a driver population unfamiliar with the area. Ref. Page 3-11. The real world free flow capacity of the facility is determined by applying the following formula:

$$Sf_i = MSF_i \text{ x } N \text{ x } f_w \text{ x } f_{HV} \text{ x } f_p$$

SFi = service flow rate for LOS i under prevailing roadway and traffic conditions for N lanes in one direction in vehicles per hour (vph),
MSFi = Maximum service flow rate for LOS i for N lanes in one direction (vph)
N = number of lanes in one direction of the freeway,
f_w = factor to adjust for the effects of restricted lane widths and lateral clearances,
f_{HV} = factor to adjust for the effect of heavy vehicles on the traffic stream, and
f_p = factor to adjust for the effect of recreational or unfamiliar driver populations.

The following discussion describes the procedures to adjust maximum freeway capacity in terms of passenger cars per lane per hour to real world mixed vehicle conditions when there are trucks in the traffic stream (the most commonly encountered condition). For adjustments due to restricted lane width, reduced lateral clearance, and other adjustment factors, refer to chapter 3 (page 3-12) of the HCM.

The capacity adjustment due to the presence of trucks is based on the fact that trucks are larger than passenger vehicles and thus physically occupy more roadway space. Further, trucks, particularly when fully loaded, tend to be less nimble and maneuverable than passenger vehicles, especially on long steep up grades.

Truck equivalency factors are used to adjust highway capacity for the presence of trucks in the traffic stream. Truck equivalency factors are a function of the percent trucks in the traffic stream, and the degree and length of the maximum vertical grade on the facility. Table 3. 4 reproduces the truck equivalency factors for various combinations of percent trucks, grades, and grade lengths found in Table 3-4 of the 1994 HCM.

Table 3. 4 Truck Equivalency Factors.

Grade %	Length (Miles)	Percent Trucks and Buses								
		2	4	5	6	8	10	15	20	25
<2	All	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
2	0.00 - 0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	0.25 - 0.50	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	0.50 - 0.75	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	0.75 - 1.00	2.5	2.0	2.0	2.0	1.5	1.5	1.5	1.5	1.5
	1.00 - 1.50	4.0	3.0	3.0	3.0	2.5	2.5	2.0	2.0	2.0
	> 1.50	4.5	3.5	3.0	3.0	2.5	2.5	2.0	2.0	2.0
3	0.00 - 0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	0.25 - 0.50	3.0	2.5	2.5	2.0	2.0	2.0	2.0	1.5	1.5
	0.50 - 0.75	6.0	4.0	4.0	3.5	3.5	3.0	2.5	2.5	2.0
	0.75 - 1.00	7.5	5.5	5.0	4.5	4.0	4.0	3.5	3.0	3.0
	1.00 - 1.50	8.0	6.0	5.5	5.0	4.5	4.0	4.0	3.5	3.0
	> 1.50	8.5	6.0	5.5	5.0	4.5	4.5	4.0	3.5	3.0
4	0.00 - 0.25	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	0.25 - 0.50	5.5	4.0	4.0	3.5	3.0	3.0	3.0	2.5	2.5
	0.50 - 0.75	9.5	7.0	6.5	6.0	5.5	5.0	4.5	4.0	3.5
	0.75 - 1.00	10.5	8.0	7.0	6.5	6.0	5.5	5.0	4.5	4.0
	>1.00	11.0	8.0	7.5	7.0	6.0	6.0	5.0	5.0	4.5
5	0.00 - 0.25	2.0	2.0	1.5	1.5	1.5	1.5	1.5	1.5	1.5
	0.25 - 0.33	6.0	4.5	4.0	4.0	3.5	3.0	3.0	2.5	2.0
	0.33 - 0.50	9.0	7.0	6.0	6.0	5.5	5.0	4.5	4.0	3.5
	0.50 - 0.75	12.5	9.0	8.5	8.0	7.0	7.0	6.0	6.0	5.0
	0.75 - 1.00	13.0	9.5	9.0	8.0	7.5	7.0	6.5	6.0	5.5
	> 1.00	13.0	9.5	9.0	8.0	7.5	7.0	6.5	6.0	5.5
6	0.00 - 0.25	4.5	3.5	3.0	3.0	3.0	2.5	2.5	2.0	2.0
	0.25 - 0.33	9.0	6.5	6.0	6.0	5.0	5.0	4.0	3.5	3.0
	0.33 - 0.50	12.5	9.5	8.5	8.0	7.0	6.5	6.0	6.0	5.5
	0.50 - 0.75	15.0	11.0	10.0	9.5	9.0	8.5	8.0	7.5	6.5
	0.75 - 1.00	15.0	11.0	10.0	9.5	9.0	8.5	8.0	7.5	6.5
	> 1.00	15.0	11.0	10.0	9.5	9.0	8.5	8.0	7.5	6.5

Inspection of Table 3. 4 clearly shows that truck equivalency factors increase as grades and grade length increase. The impact of grade and grade length diminishes as the percent of trucks in the traffic stream increase. It is also clear from Table 3. 4 that, at grades of less than 2 percent, the truck equivalency factor is 1.5 regardless of the length of the grade or the percent of trucks in the traffic stream. Since the example problem being used throughout this chapter by definition has less than 2% grades, the truck equivalency factor will be 1.5.

Table 3. 5 and Table 3. 6 are maximum freeway capacity (mixed vehicles per lane per hour) "look up" tables based on truck equivalency factor and percent trucks in the traffic stream. Table 3. 5 is for 2 lane directional freeways and Table 3. 6 is for 3 or more lane directional freeways. Table 3. 6 is the appropriate table for the example problem. Inspection of Table 3. 6 (6 or more lane facility) with a truck equivalency factor of 1.5 and future year percent trucks of 11%, reveals a free flow capacity of 2,180 vehicles per lane per hour, or 6,540 vehicles per hour (vph) for all 3 lanes.

Table 3. 5 Maximum Mixed Vehicle Traffic Capacities for Trucks in the Traffic Stream (4 Lane Facilities).

%	Truck Equivalency Factor									
Trucks	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0
0.0%	2,200	2,200	2,200	2,200	2,200	2,200	2,200	2,200	2,200	2,200
2.0%	2,178	2,157	2,136	2,115	2,095	2,075	2,056	2,037	2,018	2,000
4.0%	2,157	2,115	2,075	2,037	2,000	1,964	1,930	1,897	1,864	1,833
5.0%	2,146	2,095	2,047	2,000	1,956	1,913	1,872	1,833	1,796	1,760
6.0%	2,136	2,075	2,018	1,964	1,913	1,864	1,818	1,774	1,732	1,692
8.0%	2,115	2,037	1,964	1,897	1,833	1,774	1,719	1,667	1,618	1,571
10.0%	2,095	2,000	1,913	1,833	1,760	1,692	1,630	1,571	1,517	1,467
12.0%	2,075	1,964	1,864	1,774	1,692	1,618	1,549	1,486	1,429	1,375
14.0%	2,056	1,930	1,818	1,719	1,630	1,549	1,477	1,410	1,350	1,294
15.0%	2,047	1,913	1,796	1,692	1,600	1,517	1,443	1,375	1,313	1,257
16.0%	2,037	1,897	1,774	1,667	1,571	1,486	1,410	1,341	1,279	1,222
18.0%	2,018	1,864	1,732	1,618	1,517	1,429	1,350	1,279	1,215	1,158
20.0%	2,000	1,833	1,692	1,571	1,467	1,375	1,294	1,222	1,158	1,100
22.0%	1,982	1,803	1,654	1,528	1,419	1,325	1,243	1,170	1,106	1,048
24.0%	1,964	1,774	1,618	1,486	1,375	1,279	1,196	1,122	1,058	1,000
25.0%	1,956	1,760	1,600	1,467	1,354	1,257	1,173	1,100	1,035	978

Table 3. 6 Maximum Mixed Vehicle Traffic Capacities for Trucks in the Traffic Stream (6 or more Lanes).

%	Truck Equivalency Factor									
Trucks	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0
0.0%	2,300	2,300	2,300	2,300	2,300	2,300	2,300	2,300	2,300	2,300
2.0%	2,277	2,255	2,233	2,212	2,190	2,170	2,150	2,130	2,110	2,091
4.0%	2,255	2,212	2,170	2,130	2,091	2,054	2,018	1,983	1,949	1,917
5.0%	2,244	2,190	2,140	2,091	2,044	2,000	1,957	1,917	1,878	1,840
6.0%	2,233	2,170	2,110	2,054	2,000	1,949	1,901	1,855	1,811	1,769
8.0%	2,212	2,130	2,054	1,983	1,917	1,855	1,797	1,742	1,691	1,643
10.0%	2,190	2,091	2,000	1,917	1,840	1,769	1,704	1,643	1,586	1,533
12.0%	2,170	2,054	1,949	1,855	1,769	1,691	1,620	1,554	1,494	1,438
14.0%	2,150	2,018	1,901	1,797	1,704	1,620	1,544	1,474	1,411	1,353
15.0%	2,140	2,000	1,878	1,769	1,673	1,586	1,508	1,438	1,373	1,314
16.0%	2,130	1,983	1,855	1,742	1,643	1,554	1,474	1,402	1,337	1,278
18.0%	2,110	1,949	1,811	1,691	1,586	1,494	1,411	1,337	1,271	1,211
20.0%	2,091	1,917	1,769	1,643	1,533	1,438	1,353	1,278	1,211	1,150
22.0%	2,072	1,885	1,729	1,597	1,484	1,386	1,299	1,223	1,156	1,095
24.0%	2,054	1,855	1,691	1,554	1,438	1,337	1,250	1,173	1,106	1,045
25.0%	2,044	1,840	1,673	1,533	1,415	1,314	1,227	1,150	1,082	1,022

Queue Dissipation Rates

Capacity during queue dissipation is less than the capacity for free flow conditions, even though the lanes are unrestricted (ref: HCM page 2-29). The reduction can easily be as much as 200 vehicles per hour. According to the Highway Capacity manual "various observations of freeway queue departure rates range from as low as 1500 pcphpl to as high as 2000 pcphpl". This implies that a separate and distinct temporary "dissipation capacity" rate exists after a work zone is removed.

Removal of restrictions in front of a queue can also be analyzed similar to the dissipation of a queue at a red traffic signal. That is, the first cars move out rather slowly while follow on cars take a little less time and finally stabilize at a saturation flow rate further back in the queue.

Observed saturation flow rates (capacity) departing from traffic signals are given in Table 2-13 on page 2-32 of the 1994 HCM and are shown here in Table 3. 7.

Table 3. 7 Observed Saturation Flow Rates per Hour of Green Time.

1,470	1,572	1,651	1,682	1,785	1,791
1,832	1,840	1,875	1,827	1,896	1,905
1,910	1,936	1,937	2,000	2,000	-
Average					1,818
Standard Deviation					144

As noted earlier, various observations of freeway queue departure rates range from as low as 1500 pcphpl to as high as 2000. Using an average of 1818 with a standard deviation of 144 from analysis of the traffic signal analogy above, there is a 68% probability that the queue dissipation rate is somewhere between 1674 and 1962. Alternately there is a 95.5% probability that the queue dissipation rate is some where between 1530 and 2106. Further discussion of queue departure rates can be found on page 6-7 of the HCM.

Using a 50% reliability, the queue dissipation capacity selected for the example problem is 1,818 vehicles per lane. With 3 lanes open, total dissipation capacity becomes 5,454 vehicles per hour.

Work Zone Capacity

Traffic capacity in the work zone can be estimated from research on the capacity associated with various lane closures on multilane facilities. Table 3. 8 reflects observed work zone mixed vehicle flow capacities at several real world work zones under several lane closure scenarios(6) .

Table 3. 8 Measured Average Work Zone Capacities (ref. HCM Page 6-11).

Directional Lanes		Number of Studies	Average Capacity	
Normal Operations	Work Zone Operations		Vehicles Per Hour	Vehicles per Lane per Hour
3	1	7	1,170	1,170
2	1	8	1,340	1,340
5	2	8	2,740	1,370
4	2	4	2,960	1,480
3	2	9	2,980	1,490
4	3	4	4,560	1,520

Table 3. 8 indicates, for example, that a three lane directional facility with one lane closed and two lanes opened to traffic will have a total average work zone capacity of 2,980 vehicles per hour and only 1,490 vehicles per lane per hour. As the 1,490 vehicles per lane per hour represent the mean capacity, it incorporates a 50 percent reliability factor (i.e. half of the time the capacity will be greater than 1,490 and half the time less than 1,490). The capacity ranges observed for the average work zone capacities in Table 3. 8 are shown in Figure 3. 3. This same data is plotted as a descending cumulative probability distribution in Figure 3. 4

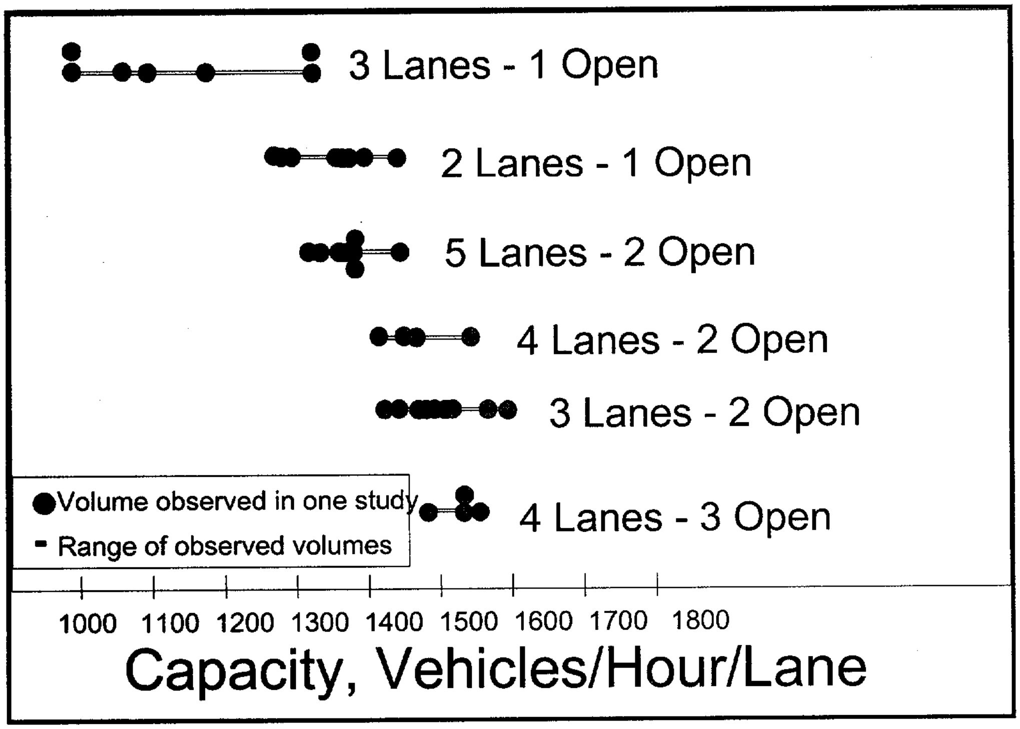

Figure 3. 3 Range of Observed Work Zone Capacities.

Figure 3. 4 is used to incorporate a reliability factor in the value selected for the work zone capacity. Figure 3. 4 is used by selecting the desired percent reliability factor from the Y axis, then intersecting the appropriate work zone situation, and estimating the corresponding capacity. This number represents the adjusted work zone directional mixed vehicle flow capacity per lane for the reliability factor selected.

For the example problem, an 80% reliability factor will be used to determine work zone capacity. By entering the figure at an 80% reliability and intersecting the curve for a 3 lane directional facility with one lane closed, the work zone capacity, determined by inspection, is approximately 1,415 vehicles per lane or 2,830 vehicles per hour. Using an 80% reliability is roughly equivalent to saying that the work zone capacity will be at least equal to 2,830 vehicles per hour 80% of the time. On the other hand it also means that the capacity of the work zone will be less than 2,830 for 20% of the time (1 day out of 5 days).

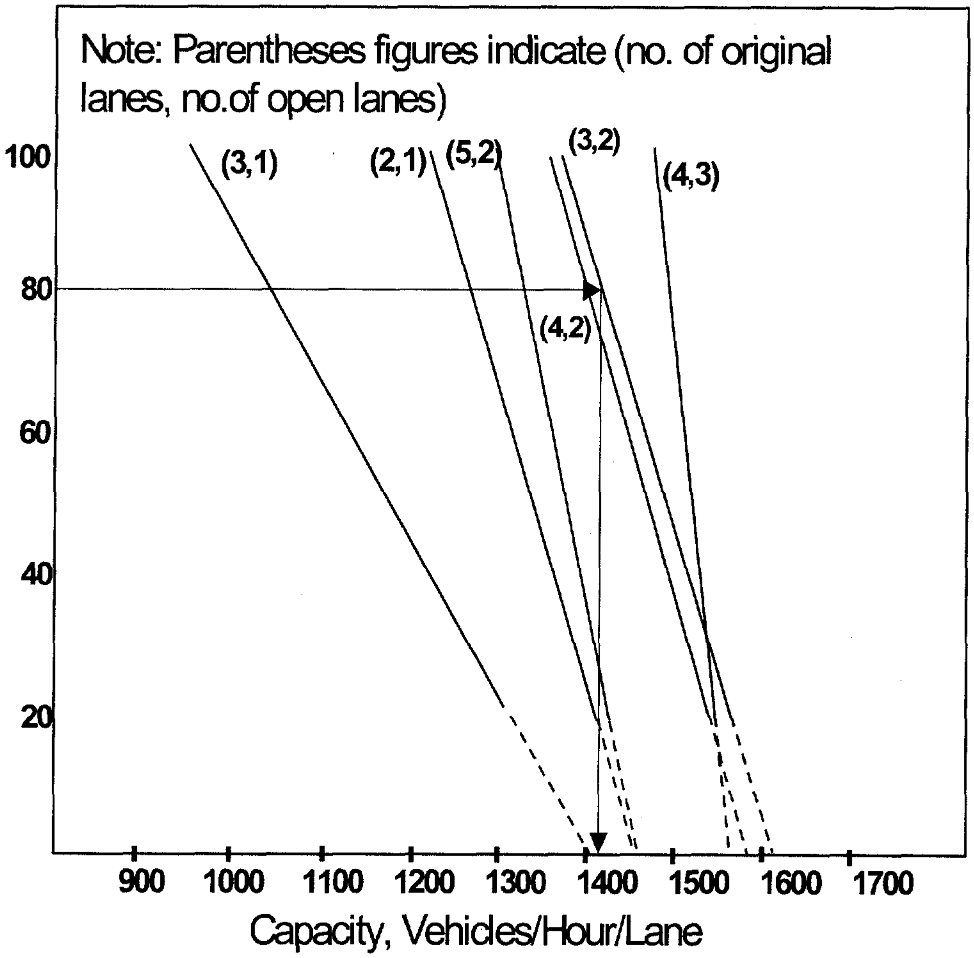

Figure 3. 4 Cumulative Distribution of Observed Work Zone Capacities. (7)

3.7.4 Identify the User Cost Components

With the roadway capacities established, the forth step is to compare the roadway capacity with the hourly demand for the facility. One of the most difficult problems in analyzing work zone user costs is keeping track of all the input values and the resulting computations. One of the most effective methods of keeping track is to set the problem up in a microcomputer spreadsheet software program. Most of the tables that follow were created in a spreadsheet software program. Table 3. 9 is such a table. It provides a convenient way to compare capacity and hourly demand and forms the basis for determining the user cost components that come into play.

Table 3. 9 Work Zone Analysis Matrix.

AADT 134,615			Queue Rate	Num. of Queued Vehicles	Lanes Open	Operating Conditions	Cost Factors
Hour	**Demand**	**Capacity**					
(a)	**(b)**	**(c)**	**(d)**	**(e)**	**(f)**	**(g)**	**(h)**
0 – 1	856	2,830	-1,974	0	2	Free Flow WZ in place No Queue	Free Flow Only (3) Costs
1 – 2	614	2,830	-2,216	0			
2 – 3	509	2,830	-2,321	0			
3 – 4	350	2,830	-2,480	0			
4 – 5	405	2,830	-2,425	0			
5 – 6	961	6,540	-5,579	0	3	Free Flow No WZ No Queue	No Costs
6 – 7	2,540	6,540	-4,000	0			
7 – 8	4,200	6,540	-2,340	0			
8 – 9	3,477	6,540	-3,063	0			
9 –10	3,150	2,830	320	320	2	Forced Flow WZ in place Queue Exists	Free Flow And Queuing (7 costs)
10 –11	3,417	2,830	587	907			
11 –12	3,639	2,830	809	1,715			
12 –13	3,769	2,830	939	2,654			
13 –14	3,837	2,830	1,007	3661			
14 –15	4,051	2,830	1,221	4881			
15 –16	4,725	5,454	-729	4152	3	Forced Flow No WZ Queue Exists	Queuing Only (4 costs)
16 –17	5,849	5,454	395	4548			
17 –18	6,865	5,454	1,411	5959			
18 –19	4,289	5,454	-1,165	4794			
19 –20	2,730	5,454	-2,724	2070			
20 –21	2,354	2,830	-476	1594	2	Forced Flow WZ in place Queue Exists	Free Flow & Queue (7 Costs)
21 –22	1,998	2,830	-832	762			
22 –23	1,610	2,830	-1,220	0			
23 –24	1,259	2,830	-1,571	0		No Queue	Free Flow
Total	67,453						

In Table 3. 9, the directional hourly travel demand determined in Table 3. 3 on page 54, is shown in column (b), while the capacities just determined are shown column (c). The capacities shown include:

(1) a work zone capacity of 2,830 vph when the work zone is in place (midnight to 5 AM, 9 AM to 3 PM, and once again from 8 PM to midnight),

(2) a free flow capacity of 6,540 vph during free flow operation (5 AM to 9 AM), and

(3) a queue dissipation capacity of 5,454 vph for the 3 PM to 10 PM time period when the work zone is removed but a built up queue exists and starts to dissipate.

Since the major work zone user cost impact results from the user delay component of traversing any queue that may develop, it is critical to determine whether or not a queue will form, and if it forms, how long it will take to dissipate. To answer this question, the directional hourly demand must be compared to the available capacity **for each hour of the day**.

The "Queue Rate" in column (d) is the difference between hourly capacity of the facility and the unrestricted hourly demand (capacity minus demand) during each hour of the day. The queuing rate is the hourly rate at which vehicles accumulate to, or, if negative, dissipate from any queue that may exist. Column (e), on the other hand, represents the cumulative number of vehicles "backed up" in the queue at the end of each hour.

To assist in the analysis, several additional columns are temporarily included in Table 3. 9:

(1) column (f) indicates the number of lanes open to traffic

(2) column (g) is used to describe the facilities operating conditions (free or forced flow)

(3) column (h) describes the user cost factors that apply during each hour of the day.

The most comprehensive approach in analyzing Table 3. 9, would be to conduct an hour by hour analysis. The analysis can be significantly simplified by grouping and analyzing hours of the day with similar operating characteristics. The analysis below centers around 5 periods of the day (one is repeated) with basically similar operating characteristics.

Period 1

Inspection of Table 3. 9 shows the work zone is in place from midnight until 5 AM and that capacity is restricted to 2,830 vph. As traffic demand is lower than capacity, the facility operates under free flow conditions. There is no queue and no vehicles have to stop. Under these conditions there are three free flow user costs due to the work zone; the VOC and delay cost of the speed change associated with slowing down for the work zone, and the delay cost of traversing the work zone at a reduced speed.

Period 2

From 5 to 9 AM the work zone is removed and, with no built up queue present, capacity increases to the free flow rate of 6,540 vph and exceeds demand throughout the period. Under these conditions, there's no queue and no work zone and therefore there are no work zone or queue related user cost.

Period 3

At 9 am when the work zone is reestablished, capacity falls to 2,830 vph until 3 PM when the work zone is removed for the evening rush hours. During this period demand exceeds capacity and a queue forms. By 10 AM the queue grows to 320 vehicles and continues to grow to 4,881 vehicles by 3 PM when the work zone is again removed. During the 9 AM to 3 PM period, there are the 3 free flow costs (speed change VOC and delay cost, and delay in traversing the work zone at reduced speed) due to the work zone as well as the 4 additional "forced flow" user cost associated with queuing, (stopping VOC and delay costs, and idling VOC and delay cost of crawling through the queue).

Period 4

At 3 PM the work zone is removed. Due to the fact that there are 4,881 vehicles already queued on the roadway, capacity only increases to 5,454 vph rather than the normal free flow capacity of 6,540 vph.

From 3 to 4 PM demand is less than capacity and the queue starts to dissipate, falling to 4,152 vehicles at 4 PM. Rush hour demand from 4 to 6 PM again exceeds capacity and the queue grows again. It grows to its maximum length of 5,959 at 6 PM before dissipating over the next two hours to 2,070 vehicles at 8 PM, when the work zone is reestablished. During this entire period from 3 until 8 PM, there is no work zone and therefor no free flow user cost. There are however, queuing user costs (stopping VOC and delay costs, and idling and delay cost of crawling through the queue).

Period 5

At 8 PM when the work zone is reestablished, there is a standing queue of 2,070 vehicles. While the capacity falls to 2,830 vph, it exceeds the hourly demand and the queue continues to dissipate. The queue is finally completely dissipated somewhere around 10:30 PM. During this period, there are once again free flow user costs (speed change VOC and delay cost, and delay in traversing the work zone) due to the work zone. There are also forced flow user costs associated with queuing (stopping VOC and delay costs, and idling and delay cost of crawling through the queue).

Period 1 ... Revisited

Finally, by 11 PM the queue is completely gone and traffic flow reverts to free flow operation through the work zone for the final hour of the 24-hour period. During this last hour, only free flow user costs are incurred. The roadway operating characteristics of 11 PM to Midnight period are identical to those of Period 1 discussed earlier.

3.7.5 Quantify Traffic Affected by Each Cost Component

The next step is to quantify the number of vehicles that experience each of the cost components. For simplicity of analysis, hours of the day are aggregated into the periods with similar operating characteristics. As discussed earlier, they are hours: Midnight to 5 AM, 5 to 9 AM, 9 AM to 3 PM, 3 to 8 PM, 8 to 11 PM, and 11 PM to Midnight.

Table 3.10 is a modification of Table 3. 9. The three columns describing operating conditions have been replaced with 4 columns (f) through (i) that provide information on the number of vehicles that have to (f) traverse the work zone, (g) traverse the queue, (h) stop for the queue, and (i) those that merely have to slow down. These four columns are used to identify the number of vehicles that experience the three free flow and the four forced flow user cost components. The table is sub-divided to cluster and subtotal the hours of the day with similar operating conditions as just discussed.

Table 3.10 Expanded Work Zone Matrix.

AADT 134,615			Queue Rate	No. of Queued Vehicles	Number of Vehicles that			
Hour	Demand	Capacity			Traverse WZ	Traverse Queue	Stop 55-0-55	Slow Down 55-40-55
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)
0 – 1	856	2,830	-1,974	0	856	0	0	856
1 – 2	614	2,830	-2,216	0	614	0	0	614
2 – 3	509	2,830	-2,321	0	509	0	0	509
3 – 4	350	2,830	-2,480	0	350	0	0	350
4 – 5	405	2,830	-2,425	0	405	0	0	405
0 - 5					2,734	0	0	2,734
5 – 6	961	6,540	-5,579	0	0	0	0	0
6 – 7	2,540	6,540	-4,000	0	0	0	0	0
7 – 8	4,200	6,540	-2,340	0	0	0	0	0
8 – 9	3,477	6,540	-3,063	0	0	0	0	0
5 - 9				0	0	0	0	0
9 –10	3,150	2,830	320	320	2,830	2,830	3,150	0
10 –11	3,417	2,830	587	907	2,830	2,830	3,417	0
11 –12	3,639	2,830	809	1,715	2,830	2,830	3,639	0
12 –13	3,769	2,830	939	2,654	2,830	2,830	3,769	0
13 –14	3,837	2,830	1,007	3,661	2,830	2,830	3,837	0
14 –15	4,051	2,830	1,221	4,881	2,830	2,830	4,051	0
9 - 15					16,980	16,980	21,861	0
15 –16	4,725	5,454	-729	4,152	0	5,454	4,725	0
16 –17	5,849	5,454	395	4,548	0	5,454	5,849	0
17 –18	6,865	5,454	1,411	5,959	0	5,454	6,865	0
18 –19	4,289	5,454	-1,165	4,794	0	5,454	4,289	0
19 –20	2,730	5,454	-2,724	2,070	0	5,454	2,730	0
15 – 20					0	27,270	24,458	0
20 –21	2,354	2,830	-476	1,594	2,830	2,830	2,354	0
21 –22	1,998	2,830	-832	762	2,830	2,830	1,998	0
22 –23	1,610	2,830	-1,220	0	2,372	1,767*	1,005*	605*
20 - 23					8,032	7,427	5,357	605
23 –24	1,259	2,830	-1,571	0	1,259	0	0	1,259
24 hours	67,453				29,005	51,677	51,677	4,597

* Values shown are prorated based on the portion of the hour required to clear the queue

Vehicles that Traverse the Work Zone – Column (f)

The traffic that traverses the work zone in column (f) is generally the traffic demand on the facility during the hours the work zone is in place. Although this is the case under free flow operating conditions, under forced flow conditions, the maximum number of vehicles that can traverse the queue is limited to the capacity of the work zone, which in the example problem is 2,830 vph.

From midnight to 5 AM, the number of vehicles traversing the work zone is the demand during the period or 2,734 vehicles. During the 9 AM to 3 PM period the demand exceeds capacity and the number traversing the queue is limited to the 2,830 vph capacity of the work zone, for a total of 16,980 during the period. From 3 to 8 PM, the work zone is removed and there is no work zone to traverse.

From 8 PM until Midnight the number of vehicles is at first limited to the capacity of the work zone because, while hourly demand is less than capacity, there is a built up queue on the road way that has to dissipate. Once the queue is dissipated, some time after 10 PM, the number of vehicles traversing the work zone area then reverts to the hourly demand.

The total number of vehicles traversing the work zone over the 24 hour period, shown at the bottom of column (f), is 29,005 vehicles.

Vehicles that Traverse the Queue – Column (g)

All vehicles that approach the work zone when a physical queue exists must stop and work their way through the queue before entering the work zone. Traffic that arrives as the queue starts to develop will have a rather short queue to work through, while traffic arriving when the queue is fully developed will have a much longer queue to traverse. On the other hand, vehicles arriving as the queue is dissipating will have a continually shrinking queue length to deal with. Nonetheless, all vehicles that arrive while a queue is present must traverse it. It is important to note, that since the facility is operating under a forced flow condition, the hourly volume of vehicles traversing the queue shown in column (g) is limited to the capacity of the work zone area. This is because the only way out of the queue is through the work zone area. During the period from 9 AM until 3 PM this capacity is 2830 vph for a total of 16,980 vehicles.

During the period from 3 until 8 PM, even though the work zone is removed, the capacity only jumps to the queue dissipation rate of 5,454 vph with 3 lanes open. During this 3 – 8 PM period, a total of 27,270 vehicles traverse the queue.

Finally, from 8 PM to 10 PM, the hourly volume traversing the queue is once again limited to the 2,830 vph work zone capacity. During the 10 to 11 PM period, when the queue finally completely dissipates, the number of vehicles traversing the queue is limited to the number of vehicles queued up at 10 PM plus those vehicles arriving early in the hour before the queue is completely gone. The 1,767 vehicles shown for 10 to 11 PM represent the prorated portion. The total number of vehicles traversing the queue during this 8 – 11 PM period is 7,427.

The total number of vehicles traversing the queue over the 24 hour period, shown at the bottom of column (g), is 51,677 vehicles.

Vehicles that Stop – Column (h)

Every vehicle that encounters a physical queue must come to a complete stop before traversing the queue. Over a 24-hour period, the number of vehicles that must stop is equal to the vehicles that must transverse the queue. The number of vehicles that stop each hour (column h) do not match up directly with the number of vehicles that traverse the queue each hour (Column g). This is because the number of vehicles which come upon a queue situation and are forced to stop, is governed by demand during the hour and not by the capacity of the work zone area. Note that the number of vehicles stopped in the 10 – 11 PM time period is a prorated share of the vehicles that approach during the hour based on the portion of the hour used to totally dissipate the 762 vehicles in the queue at the start of the hour.

The total number of vehicles that must stop over the 24 hour period, shown at the bottom of column (h), is 51,677 vehicles. This is identical to the 51,677 vehicles that traverse the queue shown at the bottom of column (g).

Vehicles that Slow Down – Column (i)

Inspection of column (i) reveals that only a small portion of the daily traffic has to "just slow down" to traverse the work zone. The number of vehicles that just have to slow down prior to traversing the work zone (as compared to coming to a complete stop), are those vehicles encountering the work zone under free flow conditions. In the example being presented here, that situation exist between midnight and 5 AM and some time after 10 PM through Midnight. During these times the hourly number of vehicles that must merely slow down is equal to the demand during that hour. The number of vehicles that merely slow down during the 10 PM to 11 PM period is less than traffic demand. That's because the first part of the hour is used to dissipate the last of the queue and only the last portion of the hour is operating under free flow conditions. The 605 vehicles shown for10 to 11 PM represent a prorated portion of the hourly demand.

The total number of vehicles that must slow down over the 24 hour period, shown at the bottom of column (i), is 4,597 vehicles.

General Comment

It is important to note that, in this example, a queue develops at mid morning and remains until sometime after 10 PM. In other situations with different traffic levels, hourly distributions, and work zone configurations, it is quite possible to have a queue develop in the morning, completely dissipate, and then reappear in the afternoon. Since the number of vehicles queued would differ, the length of the queue and associated user costs would also differ and would need to be analyzed separately.

Key summary data on the traffic effected for each of the cost components from Table 3.10 is reproduced in Table 3.11.

Table 3.11 Summary Traffic Affected By Each Cost Component.

Hours	Number of Vehicles That			
	Traverse WZ	Traverse Queue	Stop 55-0-55	Slow 55-40-55
0 - 5	2,734	0	0	2,734
5 - 9	0	0	0	0
9 – 15	16,980	16,980	21,861	0
15 – 20	0	27,270	24,458	0
20 – 23	8,032	7,427	5,357	605
23 – 24	1,259	0	0	1,259
Total	29,005	51,677	51,677	4,597

3.7.6 Compute Reduced Speed Delay

Before computing actual user cost, we must know the number vehicles subjected to speed changes, the number of vehicles that stop, and the delay time through both the work zone and the queue. The number of vehicles that under go speed changes and that stop are directly related to the affected traffic, which has already been determined. Although the number of vehicles delayed through the work zone and queue area have been determined, the amount of delay can only be computed after knowing the work zone and queue area lengths and the speeds through them.

The delay time through the work zone and through the queue are computed in the same manner. In each case, the delay is determined by subtracting the time it takes to traverse either the work zone or queue length when they are present, from the time it takes to travel the same distance when they are not present. Both calculations are dependent on the length to be traversed and the appropriate travel speeds when a work zone and/or a queue are present and when they are not.

$$\text{WZ Delay} = \frac{\text{WZ Length}}{\text{WZ Speed}} - \frac{\text{WZ Length}}{\text{Upstream Speed}}$$

$$\text{Queue Delay} = \frac{\text{Queue Length}}{\text{Queue Speed}} - \frac{\text{Queue Length}}{\text{Upstream Speed}}$$

Work zone and queue delay computations for the example problem follows:

Work Zone Reduced Speed Delay

In the example used here, the following were given:

Upstream Speed = 55 mph
Work Zone Speed = 40 mph
Work Zone Length = 5.25 miles

The work zone reduced speed delay is computed using the delay formula discussed above. The results are shown in Table 3.12

Table 3.12 Work Zone Reduced Speed Delay.

Work Zone Length (Miles)	Time at 40 mph (Hours)	Time at 55 mph (Hours)	Work Zone Delay / Veh. (Hours)	Work Zone Delay / Veh. (Minutes)
5.25	0.1313	0.0955	0.0358	2.1

Queue Reduced Speed Delay

Queue reduced speed delay is computed in the same manner, however, in this case the queue speed and queue length are not known. It is therefore necessary, in this case, to determine the queue speed and queue length for each of the three analysis time periods where queues exists.

Queue Speed Calculations:

Speed through the queue can be determined by using the Forced Flow Average Speed versus Volume to Capacity (V/C) ratio graphs for level of service "F" contained in the 1994 Highway Capacity Manual. Using the volume through the queue and the "Free Flow" capacity of the facility, the volume to capacity ratio (V/C) is calculated for each period and used to find the corresponding speed. Volume and capacity information along with the V/C calculations for three different queue periods (9-3, 3-8, and 8-11) are given in Table 3.13.

The only way for traffic to exit the queue is through the work zone and therefore the volume through the queue section is limited to the capacity of the work zone. The capacity of the work zone restricts the volume to 2,830 vph during the 9 AM to 3 PM and the 8 to 11 PM periods. The queue dissipation capacity restricts volume through the queue to 5,454 vph during the 3 to 8 PM period.

The capacity of the 3-lane directional facility where the queue forms (up stream of the work zone) is the same as the capacity in the 3-lane unrestricted upstream section operating in a free flow condition just prior to the queue. That free flow capacity was determined earlier to be 6,540 vehicles per hour.

Using these values, the V/C ratios are calculated and used in conjunction with Figure 3.5 to determine the respective queue speeds shown at the bottom of Table 3.13.

Table 3.13 Queue Speed.

Factors	Daily Time Period		
	9 AM – 3 PM	3 PM – 8 PM	8 PM – 11 PM
	(a)	(b)	(c)
Volume (Queue)*	2,830	5,454	2,830
Capacity (Roadway)	6,540	6,540	6,540
V/C	0.43	0.83	0.43
Speed	8 mph	18 mph	8 mph
* This is the Volume that moves out of the queue over a one hour period			

Inspection of figure 3.5, reveals that the speed through the queue for the 9 AM - 3 PM and the 8 -11 PM time periods will be approximately 8 mph based on the computed 0.43 V/C ratio. The speed for the 3 – 8 PM period is approximately 18 mph based the computed 0.83 V/C ratio.

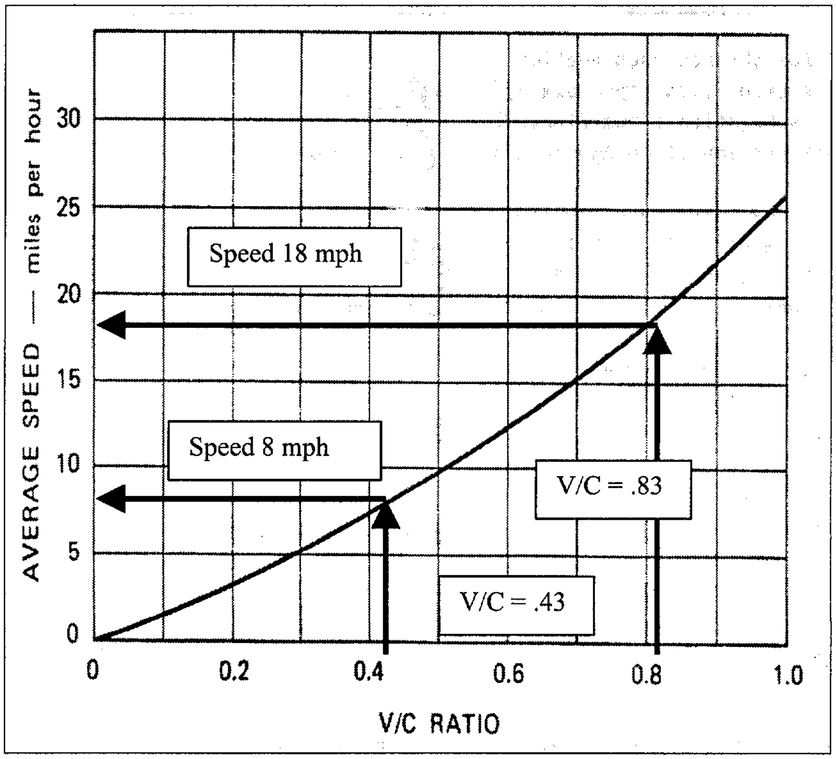

Figure 3. 5 Average Speed versus V/C Ratio (Level of Service "F"). [7]

Queue Length Calculations:

The queue length varies throughout the day as directional hourly demand and capacity through the work zone section change. The number of vehicles in the queue starts out small when it first forms around 9 AM, grows to a maximum at about 6 PM, and trails off to nothing as it totally dissipates some time after 10 PM. Some vehicles face very short delays, while others face considerable delays. Figure 3. 6 graphically displays the growth and disipation of the number of queued vehicles over the course of the day.

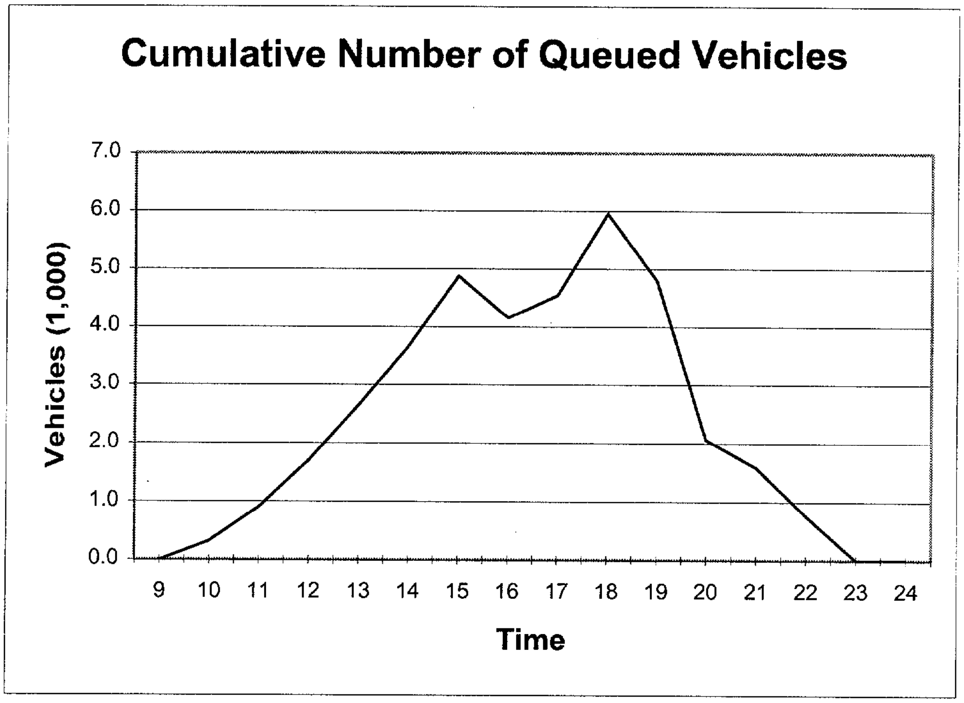

Figure 3. 6 Queued Vehicle Growth and Dissipation Over Time

Queue delay computations are based on the average queue length. This can be done by analyzing individual queue periods (periods short as one hour if desired). Alternatively, simplifying assumptions on uniform queue growth and dissipation rates can greatly reduce the number of necessary delay time computations. Assuming that the queue length grows and dissipates uniformly, the average queue length can be determined by taking 1/2 of the calculated maximum queue length. However, for the example problem, average queue lengths for the three queue periods will be computed by averaging the average hourly queue lengths over the queue period for each queue period. The average number of vehicles in the queue for each hour are computed by averaging the number of queued vehicles at the beginning and end of each hour. The computed hourly average number of queued vehicle is represented graphically in Figure 3. 7.

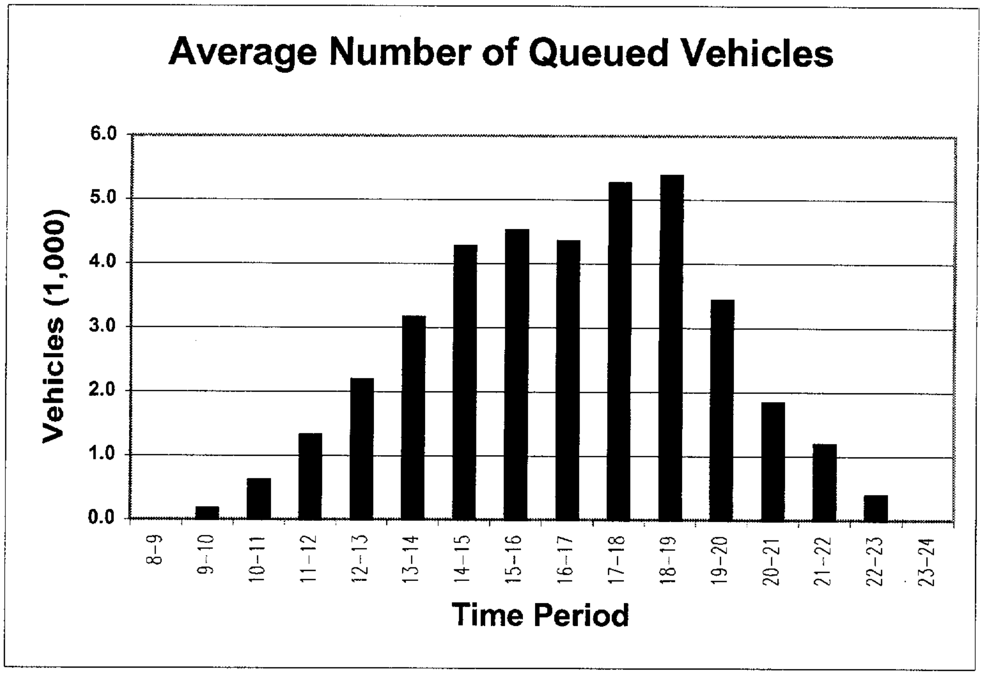

Figure 3. 7 Average Number of Queued Vehicles in Each Hour

The hourly queue length is computed by dividing the average number of vehicles in the queue during the hour by the change in traffic density between the upstream free flow section and the queue section during that hour. Traffic density is, by definition, the number of vehicles on a mile of roadway (vehicles/mile). It can be computed by dividing vehicle flow through the section (vph) by the average speed through the section (mph). Dividing traffic flow (vehicles per hour) by traffic speed (miles per hour) produces units of vehicles per mile [(vehicles/hour) / (vehicles/mile) = miles].

Traffic densities and queue lengths are computed in Table 3.14. Column (a) is the time at which the queue length is being calculated. The upstream and queue volumes (columns b and c) for the times shown in column (a) are taken from Table 3.10. The queue speeds (columns d) are taken directly from Figure 3. 5. The upstream speed (column e) was given as 55 mph in the problem definition.

The density of the queue (column f) is determined by dividing the queue flow in column (b) by the queue speed from column (d). The density of the upstream section (column g) is determined by dividing the upstream demand in column (c) by the upstream speed from column (e). The change in traffic density in column (h) is just the difference in the upstream and queue traffic densities (column f minus g).

The average number of queued vehicles (column i) is the average of the number of vehicles queued at the beginning and end of each hour. Finally, the queue length in miles (column j) is computed by dividing the number of queued vehicles (column i) by the change in traffic density (column h).

Table 3.14 Average Queue Length Calculations

Time	Volume		Speed		Density			Average No. of Queued Vehicles	Queue Length (miles)
	Through Queue	Up-stream Of Queue	In Queue	Up-stream of Queue	In Queue	Up-stream of Queue	Change		
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)
					(b / d)	(c / e)	(f - g)		(i/h)
9-10	2,830	3,477	8	55	354	63	291	160	0.55
10-11	2,830	3,150	8	55	354	57	296	613	2.07
11-12	2,830	3,417	8	55	354	62	292	1,311	4.50
12-13	2,830	3,639	8	55	354	66	288	2,185	7.60
13-14	2,830	3,769	8	55	354	69	285	3,158	11.07
14-15	2,830	3,867	8	55	354	70	284	4,271	15.04
Average for the 9-15 Period (9 AM - 3 PM)								1,950	6.80
15-16	5,454	4,051	18	55	303	74	229	4,517	19.70
16-17	5,454	4,725	18	55	303	86	217	4,350	20.04
17-18	5,454	5,849	18	55	303	106	197	5,253	26.71
18-19	5,454	6,865	18	55	303	125	178	5,376	30.17
19-20	5,454	4,289	18	55	303	78	225	3,432	15.25
Average for the 15-20 Period (3 PM - 8 PM)								4,586	22.37
20-21	2,830	2,730	8	55	354	50	304	1,832	6.02
21-22	2,830	2,354	8	55	354	43	311	1,178	3.79
22-23	2,830	1,998	8	55	354	36	317	381	2.40
Average for the 20-23 Period (8 PM to 11 PM)								1,130	3.67

The queue lengths in Table 3.14 are grouped by analysis period (i.e. 9-15, 15-20, and 20-23) in order to determine the average queue length for the period. The average queue length for each of the periods is computed by averaging the queue length at the beginning of the period with the values at the end of each hour of the period.

For the 9 AM to 3 PM period, the queue length is 0.00 miles at 9 AM and continually builds to a maximum length of 15.04 miles at 3 PM. The average queue length over the 9 AM to 3 PM period is 6.80 miles. For the 3 PM to 8 PM period, the queue length starts out at 15.04 miles, which is the same as the queue length at the end of the pervious period. The queue length continues to grow to a maximum of 30.17 miles at 6 PM. This happens in spite of the fact that the work zone is not in place during this entire 3 PM to 8 PM period. From 6 to 8 PM the traffic demand diminishes and the queue length shortens to 15.25 miles by 8 PM. The average queue length over the period is 22.37 miles. Finally, during the 8 to 11 PM period, the queue length starts out at 15.25 miles at 8 PM and drops to 0 .0 miles some-where between 10 and 11 PM. The average queue length over the period is 3.67 miles.

Queue Length Calculations – Alternate Approach

A simplified approach to calculating queue lengths entails assigning the number of queued vehicles to the available lanes and multiplying the number of vehicles per lane by an assumed average vehicle length that includes the space between vehicles. An example problem on page 6-7 of the 1994 HCM assumes an average vehicle length of 40 feet /vehicle. Using the same assumed value of 40 feet/vehicle and the average number of queued vehicles for each of the queue periods from Table 3.14, the average queue length are calculated in Table 3.15 using this alternate approach.

Table 3.15 Average Queue Length – Alternative Approach

Period	Average No. Queued Vehicles	No. Of Lanes	Average No. Queued Vehiles/Lane	Average Vehicle Length (ft/mile)	Feet Per Mile	Average Queue Length (Miles)
(a)	(b)	(c)	(d)	(e)	(f)	(b)/(c)x(e) (f)
9:00 - 15:00	1,950	3	650	40	5,280	5.0
15:00 - 20:00	4,586	3	1,529	40	5,280	11.8
20:00 - 23:00	1,130	3	377	40	5,280	2.9

The average queue lengths calculated in Table 3.15 under the alternative approach are somewhat shorter for the 9:00 AM to 3:00 PM and 8:00 to 11:00 PM when the work zone is in place). This indicates the assumed vehicle length of 40 feet is about 20% to short. The average queue length calculated for the 3;00 to 8:00 PM period when the work zone is not in place is significantly shorter (approximately 50%) than the average queue length computed using the roadway density approach. During this period the queue speed is significantly higher and it is highly unlikely that the assumed 40 feet per vehicle spacing can be maintained.

Once the maximum queue lengths and the speeds through the queue for all three queue analysis periods have been determined, the average delay for vehicles that transverse each of the queues can be computed. The average delay is computed by determining the time necessary to transverse the average queue length at the forced flow queue speed and subtracting the time it would normally take if the queue were not present. The speed without the queue present is the same as the speed in the upstream section (in this case it is given as 55 mph).

The average queue delay time for each of the three queue analysis periods is computed in Table 3.16. The travel times shown in column (c) are computed by dividing the average queue length in column (a) by the queue speed in column (b). The travel times for column (d) are computed by dividing the average queue length in column (a) by the 55 mph up stream speed. The average queue delay times shown in column (e) and (f) are determined by subtracting the travel time necessary to traverse the queue length (columns (c) minus (d)).

Table 3.16 Average Queue Delay Time.

Period	**Average Queue Length (Miles)**	**Queue Speed (Miles/Hour)**	**Time (hours)**		**Average Queue Delay per Vehicle**	
			@ Queue Speed (a/b)	**@ 55 mph (a)/55**	**Hours (c-d)**	**Minutes (c-d)**
	(a)	**(b)**	**(c)**	**(d)**	**(e)**	**(f)**
9 – 15	6.80	8	0.8505	0.1237	0.7268	43.61
15 – 20	22.37	18	1.2430	0.4068	0.8362	50.17
20 - 23	3.67	8	0.4589	0.0667	0.3921	23.53

The daily hours of queue delay are computed by multiplying the average queue delay time per vehicle by the number of affected vehicles in each period.

3.7.7 Select and Assign VOC Rates

Table 3.17 below is reproduced from chapter 2 and shows additional hours of delay and additional VOC (in August, 1996 dollars) associated with stopping 1000 vehicles from a particular speed and returning them to that speed for the three vehicle classes. Different factors are provided for Passenger cars and both Single Unit and Combination trucks. In addition, the last row of Table 3.17 shows the VOC rate associated with idling while stopped.

Table 3.17 Added Time and Vehicle Running Cost / 1000 Stops and Idling Costs (8/96).

Initial Speed (mph)	Added Time (Hr / 1000 Stops) (Excludes Idling Time)			Added Cost ($/1000 Stops) (Excludes Idling Time)		
	Pass. Cars	Trucks		Pass. Car	Trucks	
		Single Unit	Combination		Single Unit	Combination
5	1.02	0.73	1.10	2.70	9.25	33.62
10	1.51	1.47	2.27	8.83	20.72	77.49
15	2.00	2.20	3.48	15.16	33.89	129.97
20	2.49	2.93	4.76	21.74	48.40	190.06
25	2.98	3.67	6.10	28.67	63.97	256.54
30	3.46	4.40	7.56	36.10	80.23	328.21
35	3.94	5.13	9.19	44.06	96.88	403.84
40	4.42	5.87	11.09	52.70	113.97	482.21
45	4.90	6.60	13.39	62.07	130.08	562.14
50	5.37	7.33	16.37	72.31	145.96	642.41
55	5.84	8.07	20.72	83.47	160.89	721.77
60	6.31	8.80	27.94	95.70	178.98	798.99
65	6.78	9.53	NA	109.02	195.84	NA
70	7.25	NA	NA	123.61	NA	NA
75	7.71	NA	NA	139.53	NA	NA
80	8.17	NA	NA	156.85	NA	NA
Idling Cost ($ / vehicle-hour)				0.6927	0.7681	0.8248

While this table is designed to determine stopping cost, it can also be used to determine the cost and time factors associated with slowing down from 55 mph to 40 mph and returning to 55 mph. This is accomplished by subtracting the cost and time factors for stopping associated with each speed from one another. For the example problem, the speed change cost and delay factors associated with going from 55 mph to 40 mph and back to 55 mph are shown on the last line of Table 3.17 (columns a through c, and columns d through e respectively).

Table 3.18 Speed Change Computations.

Initial Speed (mph)	Added Time (Hr / 1000 Stops) (Excludes Idling Time)			Added Cost ($/1000 Stops) (Excludes Idling Time)		
	Pass. Cars	Trucks		Pass. Cars	Truck	
		Single Unit	Combinations		Single Unit	Combinations
	(a)	(b)	(c)	(d)	(e)	(f)
55	5.84	8.07	20.72	83.47	160.89	721.77
40	4.42	5.87	11.09	52.70	113.97	482.21
55-40-55	1.42	2.20	9.63	30.77	46.92	239.56

3.7.8 Select and Assign Delay Cost Rates

User delay cost rates were discussed earlier in section 2.4.3. Table 2.13 is reproduced here for convenience as Table 3.19.

Table 3.19 Recommended Values of Travel Time $/Vehicle Hour (Aug 1996).

Passenger Cars	Trucks	
	Single Unit	Combinations
$ 10 - $ 13	$17 - $19	$ 21 - $ 24

While any values within the ranges shown in Table 3.19 are considered reasonable, for purposes of the example problem, the Delay rates selected correspond to the mean valued determined in Table 2. 7. The values that will be used in connection with all delay cost computations in section 3.7.10 are:

Passenger Vehicles = $ 11.58 / Vehicle Hour
Single unit trucks = $ 18.54 / Vehicle Hour
Combination trucks = $ 22.31 / Vehicle Hour

3.7.9 Assign Traffic to Vehicle Classes

At this point it is necessary to distribute the directional traffic impacted by the various cost components to the appropriate vehicle classes for each cost component. Table 3.20 lays out the matrix used to assign the overall traffic associated with each of the user cost components to the appropriate vehicle classes.

Continuing with the example problem, the total daily traffic for each user cost component is listed in column (a) of Table 3.20. The volumes listed in column (b), (c), and (d) the reflects the distribution of the overall traffic for each of the cost components to the appropriate vehicle classes. The vehicle class distribution factors used are listed in the header for each column. These percentages were developed for the example problem in section 3.7.1. They reflect the projected distributions in 1999 (the year of the work zone will be established) based on the differential growth rates assigned to passenger vehicles and trucks. Column (e) is the sum of columns (b), (c), and (d) and is just a mathematical check to assure that the individual traffic assigned to the vehicles classes total back to the original traffic volume.

Table 3.20 Affected Traffic by Vehicle Class and User Cost Component.

Cost Component	Affected Vehicles Mixed Flow	Passenger Vehicles 89%	Trucks		Total
			Single Unit 6%	Combos 5%	
	(a)	(b)	(c)	(d)	(e)
Speed Change (55-40-55)	4,579	4,092	276	230	4,597
Traverse WZ	29,005	25,814	1,740	1,450	29,005
Stopping (55-0-55)	51,677	45,993	3,101	2,584	51,677
Queue Delay$_1$ (9 –15)	16,980	15,112	1,019	849	16,980
Queue Delay$_2$ (15-20)	27,270	24,270	1,636	1,364	27,270
Queue Delay$_3$ (20-23)	7,427	6,610	446	371	7,427

3.7.10 Compute User Cost Components by Vehicle Class

Daily user costs by vehicle class for each cost component are computed by multiplying the affected traffic by the appropriate unit cost rates (either VOC or delay) for the various components. The individual costs are computed in Table 3.21 through Table 3.27.

Table 3.21 User Cost Component # 1 - Speed Change VOC (55-40-55 mph).

Vehicle Class	Affected Vehicles	Added VOC (55-40-55) $/1,000 Vehicles	Cost ($) per day	Total Costs ($) (60 Days)
	(a)	(b)	(c)	(d)
Pass Cars	4,092	30.77	126	7,554
SU Truck	276	46.92	13	777
CU Truck	230	239.56	55	3,304
Total Speed Change VOC	4,597		194	11,634

The added VOC rates used in column (b) of Table 3.21 are from the bottom line of columns (d) through (f) in Table 3.17, while the rates used in column (b) of Table 3.22 are from the bottom line of columns (a) through (c) of Table 3.17.

Table 3.22 User Cost Component # 2 - Speed Change Delay Cost (55-40-55 mph).

Vehicle Class	Affected Vehicles	Added Time (55-40-55) Hrs./1,000 Veh.	Cost ($) per Veh. Hour	Cost ($) per day	Total Costs ($) (60 Days)
	(a)	(b)	(c)	(d)	(e)
Pass Cars	4,092	1.42	$11.58	67	4,037
SU Truck	276	2.20	$18.54	11	675
CU Truck	230	9.63	$22.31	49	2,963
Total Speed Change Delay	4,597			128	7,675

Table 3.23 User Cost Component # 3 - Work Zone Reduced Speed Delay Cost.

Vehicle Class	Affected Vehicles	Added Time (Hrs. / Veh.)	Cost ($) per Veh-Hour)	Costs ($) per day	Total Cost (60 days)
	(a)	(b)	(c)	(d)	(e)
Pass Cars	25,814	0.0358	$11.58	10,700	640,012
SU Truck	1,740	0.0358	$18.54	1,155	69,296
CU Truck	1,450	0.0358	$22.31	1,158	69,489
Total WZ Reduced Speed Delay	29,005			13,013	780,796

Table 3.24 User Cost Component # 4 - Stopping VOC (55-0-55).

Vehicle Class	Affected Vehicles	Added VOC (55-0-55) Hrs / 1000 Veh.	Costs ($) per day	Total Cost (60 days)
	(a)	(b)	(c)	(d)
Pass Cars	49,993	83.47	3,839	230,341
SU Trucks	3,101	160.89	499	29,932
CU Trucks AM	2,584	178.98	462	27,748
Total Stopping VOC	51,677		4,800	288,020

Table 3.25 User Cost Component # 5 - Stopping Delay Cost (55-0-55 mph).

Vehicle Class	Affected Vehicles	Added Time (55-0-55) Hrs. / 1000 Veh.	Cost per Hour	Costs ($) per day	Total Cost (60 days)
	(a)	(b)	(c)	(d)	(e)
Pass Cars	45,993	5.84	$11.58	3,110	186,621
SU Trucks	3,101	8.07	$18.54	464	27,835
CU Trucks	2,584	20.72	$22.31	1,194	71,665
Total Stopping Delay	51,677			4,769	286,121

Table 3.26 User Cost Component # 6 - Idling VOC.

Vehicle Class	Queue Period	Affected Vehicles	Added Time (Hours)	Idle VOC Rates ($/1000Veh-Hr)	Costs ($) per day	Total Cost (60 days)
		(a)	(b)	(c)	(d)	(e)
Pass Cars	9 AM -3 PM	15,112	0.7268	692.70	7,608	456,492
	3 PM - 8 PM	24,270	0.8362		14,058	843,480
	8 PM -11PM	6,610	0.3921		1,795	107,729
	Subtotal	45,992	-	-	23,461	1,407,701
SU Trucks	9 AM -3 PM	1,019	0.7268	768.10	569	34,125
	3 PM - 8 PM	1,636	0.8362		1,051	63,053
	8 PM -11PM	446	0.3921		134	8,053
	Subtotal	3,101	-	-	1,754	105,231
CU Trucks	9 AM -3 PM	849	0.7268	824.80	509	30,536
	3 PM - 8 PM	1,364	0.8362		940	56,423
	8 PM -11PM	371	0.3921		120	7,206
	Subtotal	2,584	-	-	1,569	94,165
Total Idling VOC						1,607,097

The Idle cost rates used in column (c) of Table 3.26 are from the bottom line of Table 3.17.

Table 3.27 User Cost Component # 7 - Queue Reduced Speed Delay Cost.

Vehicle Class	Queue Period	Affected Vehicles	Added Time (Hours)	Delay Cost Rates ($/Veh-Hr)	Costs ($) per day	Total Cost (60 days)
		(a)	(b)	(c)	(d)	(e)
Pass Cars	9 AM -3 PM	15,112	0.7268	11.58	127,188	7,631,264
	3 PM – 8 PM 8	24,270	0.8362		235,010	14,100,619
	PM -11PM	6,610	0.3921		30,015	1,800,918
	Subtotal	45,992	-	-	392,213	23,532,801
SU Trucks	9 AM -3 PM	1,019	0.7268	18.54	13,728	823,681
	3 PM – 8 PM 8	1,636	0.8362		25,366	1,521,951
	PM -11PM	446	0.3921		3,240	194,382
	Subtotal	3,101	-	-	42,334	2,540,014
CU Trucks	9 AM -3 PM	849	0.7268	22.31	13,766	825,976
	3 PM – 8 PM 8	1,364	0.8362		25,437	1,526,192
	PM -11PM	371	0.3921		3,249	194,924
	Subtotal	2,584	-	-	42,452	2,547,092
Total Queue Speed Delay						28,619,907

3.7.11 Sum Total Work Zone User Costs

A master summary of all costs is shown in Table 3.28 and the percent distributions are shown in Table 3.29. The first three cost components in Table 3.28 and Table 3.29 represent the cost associated with free flow, while the remaining four cost component represent the forced flow queuing costs.

Table 3.28 Master Summary - Total (60 day) Work Zone User Cost (August 96).

User Cost Component		Passenger Cars	Trucks		Totals ($)
			Single Unit	Combination	
		(a)	(b)	(c)	(d)
1 Speed Change VOC		7,554	777	3,304	11,634
2 Speed Change Delay		4,037	675	2,963	7,675
3 WZ Reduced Speed Delay		642,012	69,296	69,489	780,796
4 Stopping VOC		230,341	29,932	27,748	288,020
5 Stopping Delay		186,621	27,835	71,665	286,121
6 Queue Idling VOC	9 AM -3 PM	456,492	34,125	30,536	1,607,097
	3 PM – 8 PM	843,480	63,053	56,423	
	8 PM -11PM	107,729	8,053	7,206	
	Subtotal	1,407,700	105,231	94,166	
7 Queue Speed Delay	9 AM -3 PM	7,631,264	823,681	825,976	28,619,907
	3 PM – 8 PM	14,100,619	1,521,951	1,526,192	
	8 PM -11PM	1,800,918	194,382	194,924	
	Subtotal	23,532,801	2,540,014	2,547,092	
Grand Totals		26,011,067	2,773,758	2,816,427	31,601,252
Grand Totals %		82.31%	8.78%	8.91%	100.0%

Table 3.29 Master Summary - Work Zone User Cost Distribution (%).

User Cost Component	Pass Cars	SU	Combination	Totals
1 Speed Change VOC	0.02	0.00	0.01	0.03
2 Speed Change Delay	0.01	0.00	0.01	0.02
3 WZ Reduced Speed Delay	2.03	0.22	0.22	2.47
4 Stopping VOC	0.73	0.09	0.09	0.91
5 Stopping Delay	0.59	0.09	0.23	0.91
6 Queue Idling VOC	4.45	0.33	0.30	5.09
7 Queue Speed Delay	74.47	8.05	8.06	90.58
Grand Totals	82.30	8.78	8.92	100.00

Examination of Table 3.28 and Table 3.29 immediately reveals that the high user costs are not a LCCA problem, but rather a traffic control problem. Further inspection reveals that over 90 percent of the user costs result from the queue delay component. An additional 5 percent is associated with the queue idling costs and another 2 percent is associated with queue stopping

VOC and delay. Therefore, approximately 97 percent of the user costs can be avoided by not allowing the queues to develop in the first place. In the example problem, if work zone operations could be limited to evening work from 7:00 PM to 7:00 am, the queuing situation could be drastically reduced, if not completely avoided. By limiting the contractor to evening work hours only, the queue cost in the 9 AM to 3 PM period would be completely eliminated and, the evening rush hour would not have to deal with the built up queue from the mid day work zone!

Other alternatives to lowering the work zone related user costs include adding capacity prior to the development of large future traffic demands, accelerating contractor performance to reduce the overall duration the work zone is in place, limiting the overall frequency of rehabilitation activities, etc.

While the numbers may appear unreasonably large, they are not out of line when compared to user costs developed for highway facilities damaged by the North Ridge, California earth quake. Estimates of the daily user costs associated with having the damaged facilities out of service are shown in Table 3.30.

Table 3.30 Average Weekday Delay – North Ridge Earth Quake.

Route	**Delay Hours**		**Delay Cost**		**Fuel Costs ($)**	**Total Costs ($)**
	Car	**Truck**	**Car @ $6 / hr**	**Truck @ $19.20 / hr**		
I-5	51,650	4,400	310,000	85,000	40,800	436,000
I-10	135,100	3,532	811,000	68,000	110,000	990,000
SR-118	31,630	1,235	189,800	23,710	24,600	238,100
Total	218,380	9,167	1,310,800	176,710	175,400	1,664,100

Source: Wesemann, Hamilton, Tabaie, and Bare 1995
Note: 30% of companies severely impacted average shipping cost increased by 8%

3.7.12 Address Circuity and Delay Costs

The final step in calculating user costs is to address circuity and crash costs. These costs are addressed in sections that follow.

3.8 CIRCUITY

Circuity is a term used to describe the additional mileage that users travel, either voluntarily or involuntarily, on a detour to avoid a highway work zone. If traffic is forced to detour, circuity costs are:

(1) the full VOC costs ($0.31 per mile) times the excess distance the detour imposes,
(2) the appropriate $/hour delay rates times the excess detour time and,
(3) the difference in crash rates times the exposure rate and cost per crash

If no formal detour is established and circuity is the result of voluntary self imposed diversions, then a consumer surplus approach must be employed.

Example Circuity Delay Calculations:

Problem Statement:

A 5-mile rural section of Parkway between interchanges will be closed for 90 days to reconstruct a bridge taken out by flash flooding. The reconstruction will require traffic to be detoured to a 10-mile section of rural 2-lane minor arterial highway. Capacity on the Minor Arterial is currently under utilized and congestion is not expected to be a major problem. AADT on the parkway is 10,000 vehicles per day.

Circuity Delay Costs

10 miles @ 30 mph = 0.333 hours per vehicle
- 5 miles @ 50 mph = 0.100 hours per vehicle
= 0.233 Additional hours per vehicle

Additional Delay Cost =

0.233 hrs/veh x 90 days x 10,000 AADT x $11.50/veh-hr = $2,411,550

Circuity VOC (Assume a VOC rate of $0.3/mile on the Parkway and
$0.35/mile on a rural 2-lane alternate)

VOC (Secondary) = 10 miles x 10,000 vpd x 90 days x $0.35 / VMT = $ 3,150,000
VOC (Parkway) = 5 miles x 10,000 vpd x 90 days x $0.30 / VMT = $ 1,350,000

Additional VOC (Circuitry) = $ 1,800,000

Total Circuity Costs	$ 2,411,550 Additional Delay Costs
	$ 1,800,000 Additional VOC Costs
	$ 4,211,550 Total Circuity

3.9 CRASH COSTS

3.9.1 General

The highway safety community has replaced the term *accident* with the term *crash* because the term accident implies that they are unavoidable, while in reality highway crashes to a large extent are avoidable. The term "*crash costs*" is used in this Interim Technical Bulletin to describe what was previously called *accident costs.*

Crash costs associated with work zones and work zone generated circuitous travel are a function of the generally higher crash rates in work zones and on alternate/detour routes than on the primary facility in the absence of work zones. Crash rates are based on the number of crashes as a function of exposure, typically vehicle miles of travel. Crash rates are commonly specified as crashes per 100 million vehicle miles of travel (100 M VMT).

Overall crash rates for the various functional classes of roadway are fairly well established. Crash rates for work zone, however, are not. While there is a limited amount of work zone crash history data, the validity of the data used to compute the crash rates is some times suspect. Some times, crashes that occur in work zone generated queues are not classified as "work zone" crashes. Probably even more importantly, most of the time it's difficult to accurately quantify the work zone exposure rate (i.e. the length of the work zone and number of hours the day and the number of days the work zone and resultant queues are in place). Further, sometimes the crash rate, while significantly higher in work zones than non-work zones, is still low enough that there aren't any crashes in a given work zone because the exposure period is just too short to allow for statistically valid results. Finally, the problem is compounded by the fact that work zones differ significantly in the way they treat maintenance of traffic. For example: some use permanent barriers, while others use cones or drums; some narrow up lanes, while others maintain lane width and shoulders, etc. Although there appears to be a general rule of thumb that indicates that crash rates in work zones run about three times the normal rate for the facility, there does not appear to be much statistically significant research data to support this rule of thumb.

With these limitations on the availability and validity of crash rates in mind, crash costs can be developed by multiplying the unit cost per crash, by the differential crash rates between work zones and non work zones, by the vehicle miles of travel during the duration of the work zone. The duration includes both the time the work zone is in place and the time that queues are present. Vehicle miles of travel can be estimated by multiplying the percent of the facility's AADT that will be effected by the work zone and resultant queues, by the length of the work zone and the number of days the work zone will be in place.

3.9.2 Overall Crash Rates

Crash costs on detours and alternate routes used by diverted traffic are generally easier to calculate because crash rates on the primary facility without a work zone and on alternate routes are generally better known. However, even when the crash rates are better known, the exposure rate may still be difficult to determine as it requires some indication or estimate of diversion to alternate routes when no formal detours are established.

Crash rates by crash type for the various roadway functional classes can be computed using crash and VMT data contained in FHWA's most current annual *Highway Statistics* report. Table 3.31 contains 1995 data on crash fatalities and non-fatal injuries listed in the Federal Highway Administration's *1995 Highway Statistics*. Table 3.32 contains *1995 Highway Statistics* data on VMT. The data in these two tables are used to generate the crash injury rates contained in Table 3.33. The overall crash injury rates shown in Table 3.33 are people injured per 100 million vehicle miles of travel and do not specifically represent work zone injury rates.

Table 3.31 1995 People Injured in Motor Vehicle Crashes by Functional Class.

Facility Type	Rural		Urban	
	Fatalities	Non Fatal Injuries	Fatalities	Non Fatal Injuries
(a)	(b)	(c)	(d)	(e)
Interstate	2,340	55,924	2,110	253,823
Other Freeways / Expressways	-	-	1,290	86,676
Other Principle Arterial	4,498	120,271	5,868	773,446
Minor Arterial	4,300	166,241	3,755	515,586
Major Collector	5,111	187,648	-	-
Minor Collector	1,565	60,143	-	-
Collectors	-	-	2,568	218,837
Local	3,910	236,441	4,455	601,383
Total	21,724	826,668	20,046	2,449,751

Source: *1995 Highway Statistic:* Tables Fl-220 and Fl 221.

Table 3.32 1995 Vehicles Miles of Travel (Millions).

Facility Type	Rural	Urban	Total
(a)	(b)	(c)	(d)
Interstate	223,382	341,528	564,910
Other Freeway/Expressway	-	151,560	151,560
Other Principle Arterial	215,567	370,338	585,905
Minor Arterial	153,028	293,272	446,300
Major Collector	186,212	-	186,212
Minor Collector	49,936	-	49,936
Collector	-	126,929	126,929
Local	105,164	205,907	311,071
Total	933,289	1,489,534	2,422,823

Source: *1995 Highway Statistics*: Table VM-202.

Table 3.33 Crash Injury Rates (People Injured per 100 Million VMT)

Facility Type	Rural		Urban	
	Fatalities	Non-Fatal Injury	Fatalities	Non-Fatal Injuries
(a)	(b)	(c)	(d)	(e)
Interstate	1.0	25.0	0.6	74.3
Other Freeways / Expressways	-	-	0.9	57.2
Other Principle Arterial	2.1	55.8	1.6	208.8
Minor Arterial	2.8	108.6	1.3	175.8
Major Collector	2.7	100.8	-	-
Minor Collector	3.1	120.4	-	-
Collectors	-	-	2.0	172.4
Local	3.7	224.8	2.2	292.1

Source: Computed from *1995 Highway Statistics* data

The crash injury rates for rural and urban areas in Table 3.33 are computed by dividing the number of rural injuries (columns b and c), and urban injuries (columns d and e) in Table 3.31 by the appropriate VMT contained in columns (b) and (c) of Table 3.32. The crash injury rates listed in Table 3.33 are the number of fatalities (column b and d) or the number of non-fatal injuries (column c and e) per 100 million VMT.

3.9.3 Work Zone Crash Rates

Limited information on work zone crashes is contained in the Federal Highway Administration December 1989 Publication No. FHWA-RD-89-210, *Construction Cost and Safety Impacts of Work Zone Traffic Control Strategies.* The study evaluated the differences in Single Lane Closure (SLC) and Two Lane Two Way Operation (TLTWO … cross over) traffic control strategies on rural four-lane divided highways. The report includes crash data from 48 construction projects in 11 States. The traffic volumes on the routes were relatively low, generally within a 10,000 to 30,000 ADT range.

The report concluded that there were no significant differences in overall crash rates between the two traffic control alternatives for the sites and volumes studied. It further concluded that there was no significant difference in total crashes rates before verses during construction for all projects. However the report did conclude that the severity (the number of fatal and non-fatal injury crashes) increased significantly during construction periods for both the SLC and TLTWO traffic control strategies.

The 1996 Fatal Accident Reporting System (FARS) data supports this finding. Column (a) of Table 3.34 lists ranges of the percent of total motor vehicle crash fatalities that occur in work zones, while column (b) lists the number of States that fall within a range.

Inspection reveals that most SHA are experiencing a disproportionate percentage of fatalities (over 1 %) occurring in work zones relative to the VMT that takes place in work zones.

Table 3.34 1996 Work Zone Motor Vehicles Crash Fatalities as a Percent of All Fatalities.

Work Zone Fatality Ratio Ranges (%)	No. of States	
(a)	**(b)**	**(c)**
0%	3	3
0.01% – 0.49%	1	9
0.50% – 0.99%	8	
1.00% – 1.49%	16	23
1.50% – 1.99%	7	
2.00% – 2.49%	6	11
2.50% - 2.99%	5	
3.00% - 3.49%	1	2
3.50% - 3.99%	1	
4.00% - 4.49%	3	3

Publication No. FHWA-RD-89-210 also includes crash rates on the study routes before and during the time that work zones were established. The crash rates provided in the report are broken out by type of work zone, type of construction work, and type of traffic control used. However, due to the limited number of projects in the study, the statistical validity of such highly disaggregated analysis is questionable. As a result only the overall work zone crash rate data for the two work zone traffic control strategies contained in the report are presented in Table 3.35 and Table 3.36.

Table 3.35 Crash Rates on SLC.

State	Route	Length (SLC)	ADT (1,000)	Annual Crashes per Mile per 10 K ADT			Crashes per 100 Million VMT		
				Before	During	Change	Before	During	Change
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)
AZ	I-10	14.1	8.0	6.881	5.802	-1.079	183.0	159.0	-24.1
KY	I-75	8.0	26.0	1.414	0.877	-0.536	38.7	24.0	-14.7
AZ	I-40	6.2	8.8	2.478	2.058	-0.419	67.9	56.4	-11.5
LA	I-20	6.1	23.5	1.133	0.850	-0.283	31.0	23.3	-7.8
NC	I-95	11.6	20.0	1.893	1.854	-0.039	59.1	50.8	-1.1
LA	I-12	14.8	21.6	1.268	1.250	-0.018	34.7	34.2	-0.5
MI	I-196	3.0	11.4	0.000	0.000	0.000	0.0	0.0	0.0
FL	I-295	7.5	26.0	3.336	3.423	0.086	91.4	93.8	2.4
OR	I-84	16.9	12.4	1.044	1.160	0.116	28.6	31.8	3.2
NC	I-40	7.8	35.0	2.419	2.569	0.150	66.3	70.4	4.1
AZ	I-10	3.3	33.0	4.506	4.692	0.186	123.5	128.5	5.1
OR	I-5	15.5	20.6	0.297	0.787	0.490	8.1	21.6	13.4
NC	I-85	7.4	30.0	2.023	2.722	0.699	55.4	74.6	19.2
OH	I-75	7.0	25.4	1.754	2.517	0.763	48.1	69.0	20.9
KY	SR-114	14.4	7.6	2.298	3.117	0.819	63.0	87.0	24.1
NC	I-85	13.1	41.3	1.803	2.667	0.864	49.4	73.1	23.7
OR	I-84	18.4	5.6	2.053	2.980	0.927	56.2	81.6	25.4
NC	I-77	9.9	32.0	1.328	2.739	1.411	36.4	75.0	38.7
LA	I-10	11.0	24.1	1.659	3.094	1.435	45.5	84.8	39.3
WV	I-79	4.2	6.2	2.657	4.971	2.313	72.8	136.2	63.4
AZ	I-8	6.1	5.9	0.939	3.381	2.442	25.7	92.6	66.9
FL	I-295	4.8	20.0	4.999	7.604	2.605	137.0	208.3	71.4
UT	I-15	16.3	4.5	3.022	5.768	2.747	82.8	158.0	75.2
NC	I-40	3.4	30.0	1.535	4.435	2.900	42.1	121.5	79.5
MI	I-96	9.9	24.5	4.317	7.935	3.618	118.3	217.4	99.1
WV	I-64	2.7	19.0	1.587	9.522	7.935	43.5	260.1	217.4
Total		243.3	522.3						
Average				2.208	3.315	1.106	60.5	90.8	30.3

Source: FHWA-RD-89-210

Columns (e) and (f) in Table 3.35 and Table 3.36 contain a computed crash rate in the form of the annual number crashes per mile of highway per 10,000 ADT for the time periods before and during which the work zones are in place. Column (g) shows the difference. The units used in the basic report (annual number crashes per mile of highway per 10,000 ADT) are converted to more commonly used units (crashes per 100 million VMT) in columns (h) through (j). Inspection of Table 3.35 and Table 3.36 reveal that the some projects had lower crash rates during the period of construction than they had during the period before the work zone was established. Further, from the bottom lines of the tables, the overall average crash rate on SLC was slightly (though not significantly) higher during work zone operations, while the overall average crash rate on TLTWO was slightly (though again not significantly) lower during work zone operations.

Table 3.36 Crash Rates on TLTWO.

State	Route	Length (SLC)	ADT (1,000)	Annual Crashes per Mile per 10 K ADT			Crashes per 100 Million VMT		
				Before	During	Change	Before	During	Change
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)
LA	I-20	7.2	23.9	6.607	2.332	-4.285	181.1	61.2	-119.8
UT	I-15	8.8	5.2	5.447	2.815	-2.631	149.2	77.1	-72.1
WV	I-64	2.1	27.0	5.495	2.931	-2.565	150.5	80.3	-70.2
LA	US-190	2.4	16.0	9.505	7.097	-2.408	260.4	194.4	-66.0
MI	I-96	12.0	12.8	5.104	3.062	-2.042	139.8	83.9	-55.9
FL	SR-95	2.8	4.9	5.173	4.803	-0.369	141.7	131.6	-10.1
MI	I-94	10.0	29.0	2.884	2.535	-0.350	79.0	69.5	-9.6
KY	WKP	5.5	4.2	2.048	1.756	-0.293	56.1	48.1	-8.0
UT	I-84	10.7	3.8	3.265	3.019	-0.246	89.5	82.7	-6.7
AZ	I-10	2.2	12.0	2.496	2.304	-0.192	68.4	63.1	-5.3
LA	I-20	8.8	13.5	1.703	1.606	-0.097	46.7	44.0	-2.7
OR	I-5	13.2	22.6	0.978	1.043	0.065	26.8	28.6	1.8
LA	I-20	4.7	27.6	1.590	1.877	0.287	43.6	51.4	7.9
MI	I-94	10.0	18.5	3.886	4.185	0.299	106.5	114.7	8.2
LA	I-59	6.6	13.0	3.314	3.615	0.301	90.8	99.0	8.2
NC	I-40	16.9	25.0	0.980	1.700	0.720	26.8	46.6	19.7
KY	I-75	4.0	23.0	1.469	2.498	1.029	40.2	68.4	28.2
OR	I-5	7.1	24.2	0.712	1.780	1.068	19.5	48.8	29.3
NC	I-40	11.7	17.0	2.050	3.415	1.365	56.2	93.6	37.4
WV	I-77	2.6	9.3	2.516	4.193	1.677	68.9	114.9	45.9
NC	US-1	1.8	15.0	2.674	4.457	1.783	73.3	122.1	48.8
MI	I-69	6.1	15.5	1.397	4.239	2.841	38.3	116.1	77.9
Total		157.1	363.0						
Average				3.062	2.864	-0.198	83.9	78.5	-5.4

Source: FHWA-RD-89-210

Work zone crash rate data is also included in the November 1995 Final report for the Joint Highway Research Project FHWA/JHRP-95/1 *An evaluation of Lane Closure Strategies for Interstate Work Zone.* Data contained in this report was based on the study of 26 Interstate 4R Projects in Indiana. Table 3.37 and Table 3.38 provide data on over all and injury crash rates.

Table 3.37 Average Overall Crash Rates

Work Zone Type	Lanes in Each Direction	Mean and Standard Deviation	Crash Rates per 10 million VMT		
			Without Work Zone	With Work Zone	Change
Cross Over	2	Mean	6.0329	8.0431	2.0102
		Standard Deviation	1.6842	3.6017	3.2005
Partial Lane Closure	2	Mean	5.5916	7.4528	1.8612
		Standard Deviation	1.4645	3.1398	3.3354
Cross Over	3	Mean	5.8278	9.3544	3.5266
		Standard Deviation	1.2350	5.9645	5.6871
Partial Lane Closure	3	Mean	7.5166	10.1006	2.5840
		Standard Deviation	1.6422	2.6940	3.4964

Source: FHWA/JHRP-95/1

Table 3.38 Average Fatal and Non-Fatal Injury Crash Rates

Work Zone Type	Lanes in Each Direction	Mean and Standard Deviation	Injury Crash Rates per 10 million VMT		
			Without Work Zone	With Work Zone	Change
Cross Over	2	Mean	1.1289	2.0746	0.9457
		Standard Deviation	0.5376	1.9380	2.1879
Partial Lane Closure	2	Mean	1.0969	2.0311	0.9342
		Standard Deviation	0.4252	1.3405	1.2684
Cross Over	3	Mean	1.5885	2.6367	1.0482
		Standard Deviation	0.3961	1.5320	1.3851
Partial Lane Closure	3	Mean	1.7641	2.1128	0.3487
		Standard Deviation	0.2829	1.0574	1.1565

Source: *FHWA/JHRP-95/1*

The reader should note that the rates in Table 3.37 and Table 3.38 are per 10 million VMT.

3.9.4 Example Crash Cost Calculations

The following example problem is used to lay out a rational approach to calculating the additional crash costs associated with a work zone generated detour.

Problem Statement:

A 5-mile rural section of Parkway between interchanges will be closed for 90 days to reconstruct a bridge taken out by flash flooding. The reconstruction will require traffic to be detoured to a 10-mile section of rural 2-lane minor arterial highway. Capacity on the Minor Arterial is currently under utilized and congestion is not expected to be a major problem. AADT on the parkway is 10,000 vehicles per day.

Solution:

Table 3.39 shows the matrix used to calculate the differential crash costs. Column (c) lists the crash rates for the facility and crash types listed in columns (a) and (b). The rates themselves are taken from columns (b) through (e) of Table 3.33. The crash fatality rate is 2.8 fatalities per 100 million VMT on rural 2-lane minor arterial highways and 2.1 on rural parkways (other principle arterial routes). The injury crash rates per 100 million VMT is 108.6 on rural 2-lane minor arterial highways and 55.8 on rural parkways (other principle arterial routes).

Table 3.39 Crash Cost Calculation Matrix.

Facility	Crash Rate		Exposure				No. of Crashes	Crash Cost Rate	Crash Costs ($)
	Type	Crash/ 100 M VMT	Vehicles per day	No. of Days	Miles	VMT (100 M)			
(a)	(b)	(c)	(d)	(e)	(f)	(g)	(h)	(i)	(j)
						(d)x (e)x(f)	(c)x(g)		(h)x(I)
Parkway	Fatality	2.1	10,000	90	5	0.045	0.0945	1,240,000	117,180
	Injury	55.8	10,000	90	5	0.045	2.5110	27,800	69,806
Subtotal	All								186,986
Detour	Fatality	2.8	10,000	90	10	0.090	0.2520	1,240,000	312,480
	Injury	108.6	10,000	90	10	0.090	9.7740	27,800	271,717
Subtotal	All								584,197
Change									397,211

In addition to the higher fatality and non-fatal injury crash rates, detoured and diversionary traffic generally have a greater crash exposure because of the generally greater travel distances. Column (g) of Table 3.39 list the exposure for the two alternative routes in units of 100 million VMT. The values listed were computed by multiplying the ADT (column d) by the number of days (column e) by the length of each alternative route (column f) and dividing the answer by 100 million.

The actual number of crashes expected (column h) are calculated by multiplying the crash rates in column (c) by the exposure in column (g).

Finally the crash cost listed in column (j) are computed by multiplying the expected number of crashes in column (h) by the crash cost rates in column (i). The crash cost rates used in column (i) were selected from the updated MicroBENCOST default crash cost rates discussed earlier in Section 2.4.4 on page 28.

Further information on work zone crash rates is available in the participant's handbook for NHI Course # 38003, Design and Operation of Work Zone Traffic Control. It discusses "Before' and "After" work zone crash rates on page 2-7. Results from Virginia and Texas are included.

Virginia data for 1991 indicates an increase of 57% on multi lane highways and 168% on two lane urban highways with the variation being a function of traffic, geometry and environment. In Texas, 1984-1988 data shows freeway construction crashes increase by 28.7% on the main line and 2.4% on frontage roads. It also shows a 37.4% increase at nighttime compared to 24.4% during the daytime. The data also suggest that the average changes in severe mainline crash rates were consistent from project to project, while other mainline crash categories varied significantly from site to site.

Computerized User Cost Models

Microcomputer software programs such as MicroBENCOST are available for conducting LCCA on routine pavement rehabilitation projects. It is an automated and improved version of the 1977 AASHTO "Red Book" on User Cost Analysis. The software program, with full documentation is available through McTrans Microcomputer Support Center at the University of Florida for about $100. QueWZ is another microcomputer software program available from McTrans (approximately $25.00) that can be used to simplify user cost computations.

There is also a microcomputer program called Delay Enhanced. Delay Enhanced is a relatively straightforward personal computer based program designed to analyze traffic related user cost associated with vehicle crashes and other roadway incidents. While the program as currently written does not lend itself to analysis of work zone traffic congestion, it being revised to incorporate analysis of work zone lane closures. The current version Delay Enhanced (Version 1.2) is available through Mr. Martin Knopp, Federal Highway Administration, Utah Division Office. His phone number is 801-963-0182.

Finally, when attempting to analyze user cost on highly complex urban freeway, the simplified procedures outlined in this chapter are not likely to be sufficient. In such cases, the analyst would do better to utilize the battery of urban planning models used by the major metropolitan planning organizations (MPOs). Traffic assignment models, run on the network level can provide better estimates of traffic diversions from the route in question and the impact of such diversions of traffic flow on alternate routes. Such models can be run to determine the network level changes in vehicle miles of travel (VMT) and hours of traffic delay with and with out the work zone in place. When network level changes in VMT and Delay are know, user cost calculations become much simpler.

CHAPTER 4 RISK ANALYSIS APPROACH

This chapter introduces a probabilistic based risk analysis approach to life cycle cost analysis (LCCA) in pavement design. This chapter combines the use of probability theory and advanced simulation techniques as the method of choice in the treatment of uncertain LCCA variables. It introduces the concept of risk and examines some of the limitations associated with the current deterministic approach. The chapter outlines a general risk analysis approach and provides an example of its application. While this chapter focuses on the use of risk analysis in a pavement design setting the principals and techniques put forth offer potential in other areas where uncertainty is an important consideration in the decision making process.

4.1 WHAT IS RISK

The concept of risk comes from the uncertainty associated with future events -- the inability to know what the future will bring in response to a given action today. Risk can be subjective or objective. Subjective risk is based on personal perception – intuitively deciding how risky a situation may be. For example, many people may feel that flying is more risky compared to driving. This perception of risk may be related to the consequences of failure, as well as, the ability (or inability) to be in control of the situation. Objective risk is based on theory, experiment, or observation. Often the facts of the situation may dispute intuitive feelings. For example, in 1996 there were 1070 aviation fatalities while in the same year there were 40,676 highway fatalities. Because people vary in their perceptions of risk, decisions incorporating risk management concepts, will depend to a large extent on the decisionmaker's tolerance for risk.

4.2 WHAT IS RISK ANALYSIS?

Risk analysis is technique that combines probability descriptions of the uncertainty of the input parameters and computer simulation to characterize the risk associated with future outcomes. It exposes areas of uncertainty typically hidden in traditional deterministic approach to LCCA, and allows the decision maker to weigh the probability of an outcome actually occurring.

4.3 THE NEED FOR RISK ANALYSIS

Many analytical models treat input variables as discrete fixed values as if their values were certain, when in fact they are not.. Economic models used in a typical LCCA are no exception. In conducting LCCA it is important to be aware of the inherent uncertainty surrounding the variables used as inputs into the analysis. Uncertainty results from the assumptions, estimates, and projections that are made for the input parameters. Table 4. 1 provides a summary of LCCA input variables and the general basis used to determine their values.

Table 4. 1 Input Variables for LCCA.

LCCA Component	Input Variable	Source
Initial and Future Agency Costs	Preliminary Engineering	Estimate
	Construction Management	Estimate
	Construction	Estimate
	Maintenance	Assumption
Timing of Costs	Pavement Performance	Projection
User Costs	Current Traffic	Estimate
	Future Traffic	Projection
	Hourly Demand	Estimate
	Vehicle Distributions	Estimate
	Dollar Value of Delay Time	Assumption
	Work Zone Configuration	Assumption
	Work Zone Hours of Operation	Assumption
	Work Zone Duration	Assumption
	Work Zone Activity Years	Projection
	Crash Rates	Estimate
	Crash Cost Rates	Assumption
NPV Present Value	Discount Rate	Assumption

The traditional approach to addressing this uncertainty in LCCA has been to ignore it. The analyst makes a series of "best guesses" of the values of each of the input variables and then computes a single "deterministic" result. The problem with this approach is that information that could improve the decision is often excluded.

In some cases a limited sensitivity analysis may be conducted whereby various combinations of inputs are selected to *qualify* their effect on analysis results. However, even with a sensitivity analysis, this deterministic approach to LCCA often conceals areas of uncertainty that may be a critical part of the decision making process. Often, stakeholders seize upon the uncertainty associated with LCCA inputs and vigorously debate the validity of the results. Traditional, deterministic based, LCCA results such as these generate endless debate over which alternative "truly" has the lowest life cycle cost. This process encourages division and unproductive debate.

The need to make strategic long-term investment decisions under short-term budget constraints force State highway agencies to incorporate risk (either implicitly or explicitly) as a criterion for judging which course of action to take. Risk analysis exposes areas of uncertainty that the decisionmaker may not be aware of and based on this new information provides the decisionmaker the opportunity to take mitigating action to decrease exposure to risk. With the emergence of user-friendly computer software, quantitative risk analysis concepts can be easily integrated into the decision making process of a State highway agency.

4.4 GENERAL APPROACH

The basic risk analysis approach in LCCA is illustrated in Figure 4. 1. It shows the basic net present value formula typically used as the economic indicator in a LCCA. As shown in Figure 4. 1, a risk analysis approach uses probabilistic descriptions of uncertain input variables (initial cost, future cost, discount rate, and year of rehabilitation) to generate a probabilistic description of results. A computer simulation, known generally as the Monte Carlo simulation, is then performed, where hundreds, even thousands, of samples are randomly drawn from each input distribution to calculate a separate "what-if" scenario. With the speed and memory of today's personal computers such iterations can be conducted in a matter of minutes or even seconds. The results generated from each what-if iteration are captured for later statistical analysis.

Similar to the inputs, risk analysis results are presented in the form of a probability distribution describing the range of possible outcomes along with a weighting of their probability of occurrence. In other words, by randomly drawing samples from the model's input distributions, the computer combines the variability, inherent to the inputs, and summarizes it in the form of a probability distribution for consideration by the decisionmaker. As a result, the decisionmaker knows not only the full range of possible values, but also the relative probability of any particular outcome occurring.

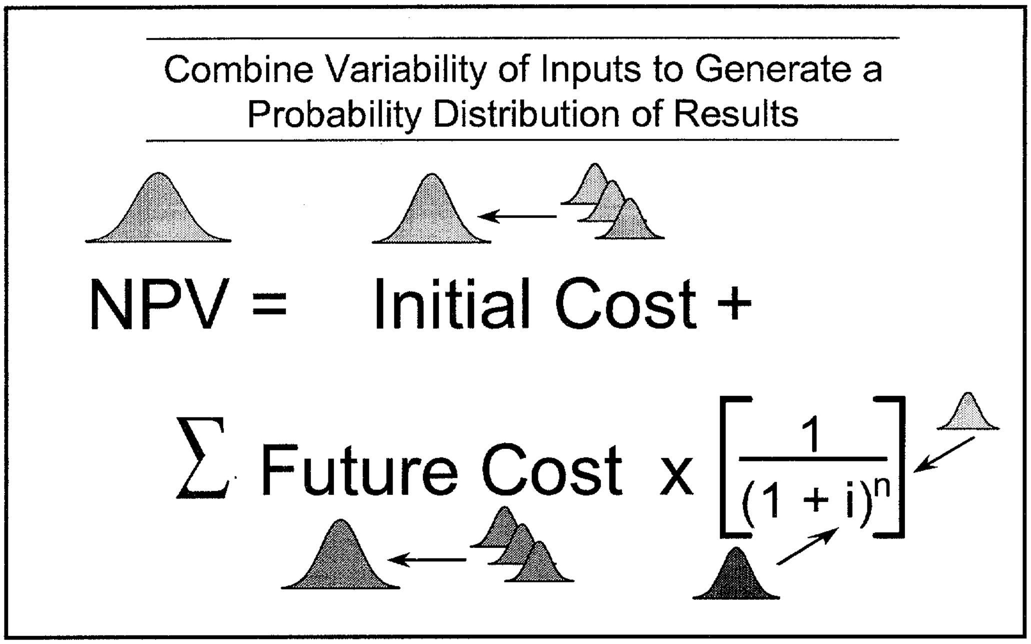
Combine Variability of Inputs to Generate a Probability Distribution of Results
NPV = Initial Cost +
$\sum$ Future Cost x $\left[\frac{1}{(1+i)^n}\right]$

Figure 4. 1 Computation of Net Present Value using Probability and Simulation.

By including all possible values for the analysis inputs in relation to their probability of occurrence, risk analysis elevates the LCCA debate, from the validity of results, to deciding best public policy. Armed with this new information, risk analysis provides the decisionmaker the opportunity to take mitigating action to decrease exposure to risk. Risk analysis provides those vested with the appropriate authority and accountability, namely executives and elected officials, the opportunity to make decisions about taking risk. (8) With the recent advances in computer technology and software development, quantitative risk analysis concepts and techniques can be easily incorporated into the decision making process.

Although the risk analysis approach is calculation intensive and would be impractical to apply by hand, the approach can be incorporated into traditional "deterministic" analysis through the use of emerging user-friendly computer software programs that incorporate advanced risk analysis concepts and simulation techniques such as the Monte Carlo. Such software can come in the form of stand-alone programs, (Microsoft Visual Basic, C++, etc.) or simple "add-ins" to spreadsheets such as Microsoft Excel or Lotus.

Due to the flexibility that spreadsheets offer in solving a wide variety of problems this technical bulletin addresses risk analysis in a spreadsheets environment. The two most often used spreadsheet add-ins risk analysis software programs are @RISK by Palisade Corporation and

Crystal Ball by Decisioneering. They are very similar in operation and capability. For presentation purpose, this technical bulletin discusses the use of @RISK in conjunction with Microsoft Excel spreadsheets.[1]

Risk analysis has been used for a number of years in other industries and, as a result, a number of approaches have evolved to conduct such analysis. The following sections outlines a general approach that may be used to conduct a risk analysis. The proposed steps are as follow:

1. Identify Structure and Logic of Problem
2. Quantify Uncertain Variables
3. Perform Simulation
4. Analyze and Interpret Results
5. Make a Consensus Decision

Each of these steps will be covered in detail in the following sections. The following example problem will be used to illustrate how risk analysis may be applied in a pavement design LCCA setting. To keep the example simple user costs are omitted.

A State highway agency wants to determine the NPV of two competing pavement design alternatives, A and B, using a probabilistic-based life cycle cost analysis. The analysis period is 35 years, routine reactive maintenance costs differences between alternatives are insignificant, and the discount rate may range from 3 to 5%. Averages and Standard Deviations for Agency Costs have been developed from an analysis of recent bid records and are shown in Table 4. 2. However, due to a lack of documented field performance data the State convened a special panel of pavement experts to develop estimates of pavement service life for each the proposed design alternatives. These estimates are shown Table 4. 3

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[1] Palisade Corporation's web address is http://www.palisade.com and can be reached at 800-432-7475. Crystal Ball's web address is http://www.decisioneering.com and can be reached at 800-289-2550.

Table 4. 2 Average and Standard Deviations for Agency Costs.

Cost Item	Alternative A		Alternative B	
	Average	**Std Dev**	**Average**	**Std Dev**
Initial Agency Cost ($ Millions)	26.5	0.75	20.0	2.5
Future Rehabilitation Cost ($ Millions)	7.0	0.5	6.0	1.0

Table 4. 3 Estimates of Pavement Service Life.

	Alternative A			Alternative B		
	Min	**Most Likely**	**Max**	**Min**	**Most Likely**	**Max**
Initial Pavement Design (Years)	20	25	30	12	15	18
Future Rehabilitation Design (Years)	10	13	15	5	7	10

4.4.1 Identify the Structure and Layout of the Problem

The first step in a risk analysis is to identify the structure and layout of the problem. This usually involves reducing the problem to its most basic elements and describing it in the form of an analytical model. Sometimes, flow charts are used to supplement the model and clarify relationships between variables. In this example problem the model is defined by the formula for NPV as shown below:

$$NPV = InitialCost + \sum FutureCosts\left(\frac{1}{(1+i)^{n}}\right)$$

This model may be programmed in a spreadsheet or a stand-alone program may be developed using Visual Basic, C+, etc.

Figure 4. 2 shows the pavement service life curves for each alternative strategy based on the mean service life values from Table 4. 3. Figure 4. 3 graphically depicts the expenditure streams for each alternative based on the mean service life values from Table 4. 3.

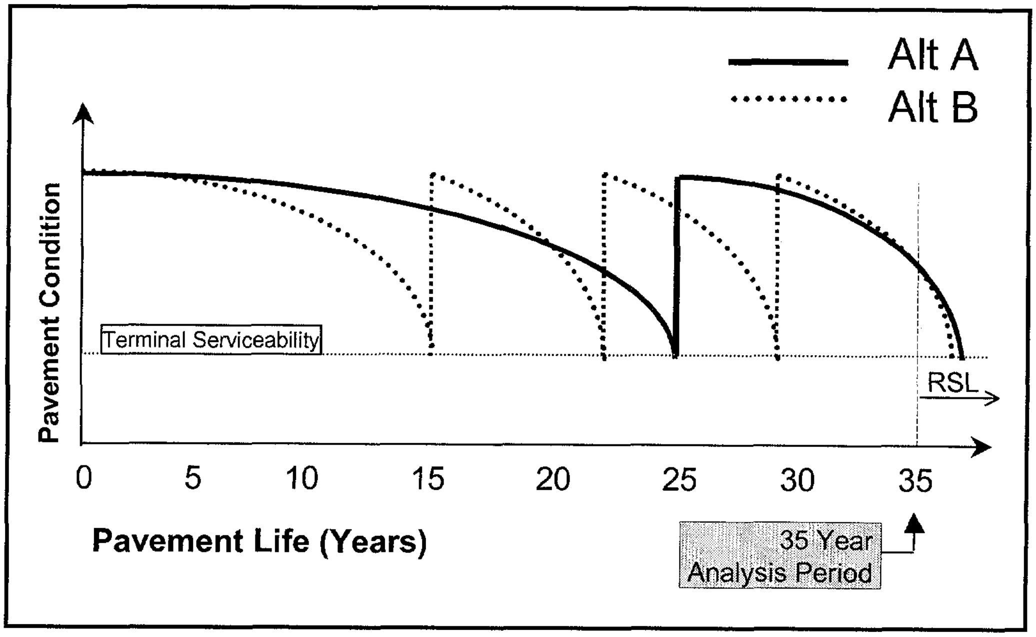


Figure 4. 2 Pavement Life Curves for Alternatives A and B.

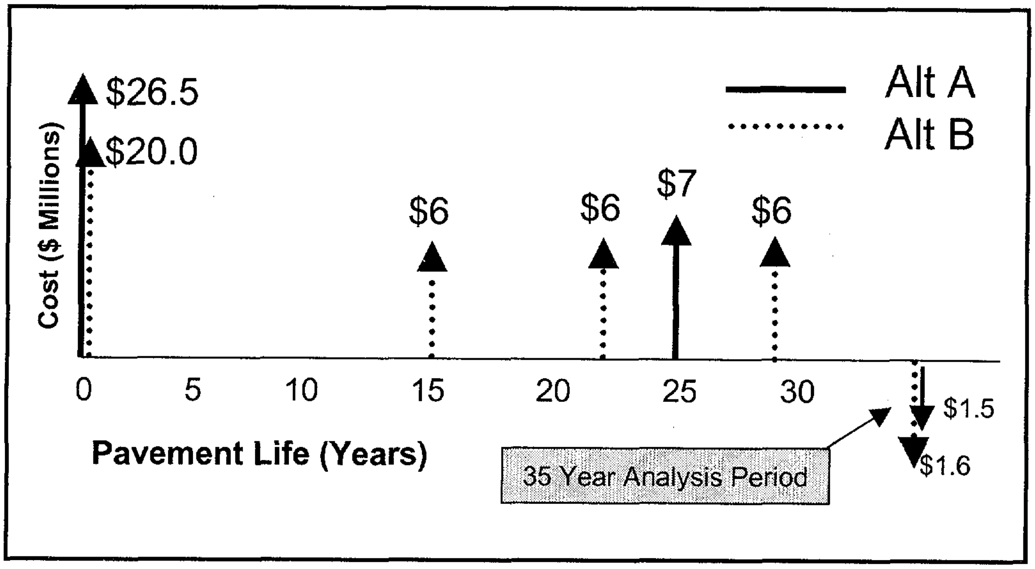


Figure 4. 3 Cash Flow Diagram for Alternatives A and B.

Based on the mean values of estimated service lives, alternatives A and B have remaining service life (RSL) at the end of the 35-year analysis period. The value of the RSL must be properly accounted for and appropriately discounted. In a deterministic LCCA treatment of the

RSL of each alternative would be relatively straightforward. In the probabilistic risk analysis approach the RSL issue becomes more complex.

The NPV model must take into account the entire range of probable pavement service lives for both the initial design and future rehabilitation designs. As such, there is a discrete probability that the last rehabilitations (as defined using mean service life values) shown in Figure 4. 2 could take place earlier than the mean service life values would indicate. At the same time there is a discrete probability that the last rehabilitations shown might well only be required at some point beyond the end of the analysis period. As a result, probabilistic based risk analysis models must account for these possibilities in determining the number, timing and RSL of future rehabilitation requirements. Typically, logical IF statements provided in most spreadsheet programs are used to facilitate discounting future costs over the entire range of probable service life values.

In developing the structure and layout of the model it is also critical to identify dependencies among uncertain variables to avoid producing incorrect results. Since user costs have been omitted in the example problem, the number of variables has been significantly reduced. The uncertain variables in this model are initial cost, future costs, discount rate, and year of rehabilitation activity. The next step is to describe the uncertainty associated with the input variables

4.4.2 Quantify Uncertain Variables

The next step is to develop probability distributions for the uncertain variables identified in the previous step. A probability distribution allows the analyst to describe the complete range of values the variable may assume and weights their likelihood of occurrence accordingly.

4.4.2.1 Types of Probability Distribution

Figure 4. 4 provides some illustrative examples of common probability distribution types in histogram form.

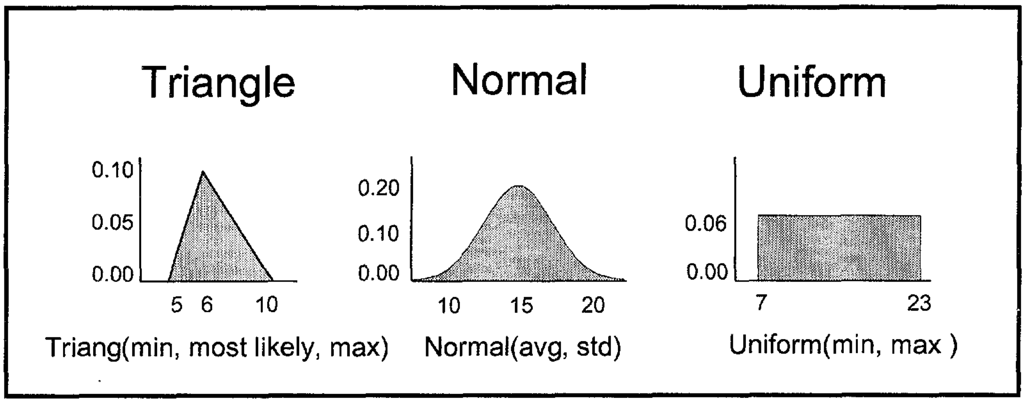


Figure 4. 4 Example Probability Distributions.

Figure 4. 4 depicts triangle, normal, and uniform distributions in histogram format. The horizontal axis provides a range of possible values and the vertical axis provides a probability weighting of the occurrence of any particular value. For the histograms shown, the probability is equal to area under the curve and the total shaded area is equal to one. For the triangular distribution the mean is given by the average of the minimum, most likely, and maximum value.

Probability distributions may also be presented in a cumulative form as shown in Figure 4. 5. In using cumulative distributions the probability is read directly off the vertical axis. For the example shown there is a 50% probability that project costs will be less than or equal to $8 million.

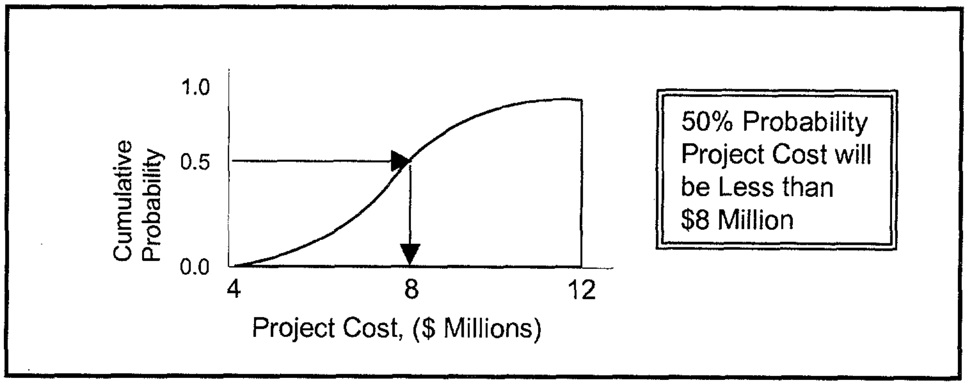


Figure 4. 5 Ascending Cumulative Probability Distribution.

4.4.2.2 Discrete and Continuous Variables

Probability distributions may be used to model discrete or continuous variables. Discrete variables are countable. Continuous variables are not. For example, time passes gradually; whereas the numbers on a digital wristwatch change abruptly. In this case, time is a continuous variable, whereas, the watch's measurement of time is not. In practice, a continuous distribution may be used to model a discrete variable, as long as the difference between allowable values is small. For example, project costs, is a discrete variable with steps of one cent. However, it may be modeled using a continuous distribution.

4.4.2.3 Developing Probability Distribution

Probability distributions that can be used to describe the variability associated with a particular input variable may be developed using either objective or subjective methods. The objective method uses hard data (such as bid price list, observed capacity, etc) while the subjective method uses expert opinion.

When existing data is available, statistical analysis packages on the market today can be used to automatically identify the probability distributions that "best fit" the data. These programs compare the more common distribution types with available data. Along with recommendations on the distribution types that best describe the variability of the data, some software packages also provide statistical indicators that describe the "goodness of fit" such as: Chi squared, Anderson-Darling (A-D), and Kolmogorov-Smirnoff (K-S). These statistics indicate how closely the probability distribution fits the data.

In the subjective method, group interviews with subject matter experts, as illustrated in Figure 4. 6, are generally used to establish the boundaries and general shape of the distributions that will be used to quantify input variabilty. This process of eliciting information from experts is similar to the well-known Delphi method. Such meetings are structured to elicit all expert opinion. Background information is provided to participants using scatter plots, trend charts, basic statistics, and histograms to summarize available data. Discussion topics include not only over all uncertainty of the input variables but also the possible inter relationships and co-dependencies among the input variables. Following up activities may include individual interviews with meeting participants to ensure all opinions are included. Formal surveys and questionnaires may also be used in the process to ensure that the distribution adequately covers the range of possible values. The importance of involving *all* stakeholders in this process to facilitate buy-in on model results cannot be overemphasized.

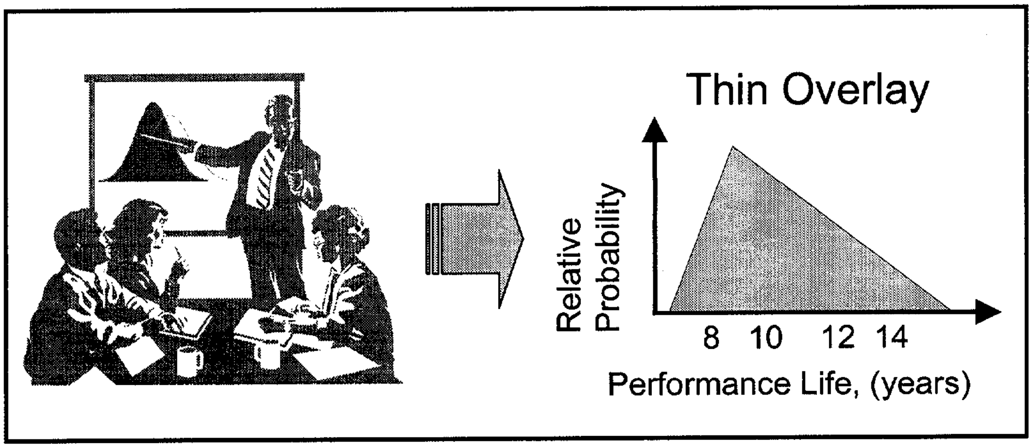


Figure 4. 6 Using Expert Opinion to Develop Probability Distributions.

The distributions most often used to model the opinion of experts include:

- ☐ **Triang** - No tail values.
- ☐ **Trigen** - Allow for tail values.
- ☐ **Normal** - If you believe the data to be normally distributed, a technique is to back into the distribution, given the mean, min and max values. Using the Empirical rule, you know that + or - two standard deviations approximate 95% of the data. The equation to determine the standard deviation is: (Max – Min) / 4.

- **General** - This distribution is very flexible, for it allows the expert task group to tailor the shape of the curve.
- **Uniform** - As a gross estimating tool. A problem with the uniform is that outside the min and max values, the probability precipitously drops to zero.
- **Discrete** - To model known probabilities or to weight expert opinions.

4.4.2.4 Selecting Distributions and Defining Parameters

Returning to the example problem, the mean values and standard deviations for agency costs provided in Table 4. 2 strongly suggests that agency costs are normally distributed as mean value and the standard deviation are the parameters necessary to describe normal distributions. When using hard data to determine the type of distribution to use, it is important to check the reasonableness of the "goodness of fit" statistics associated with the selected distribution.

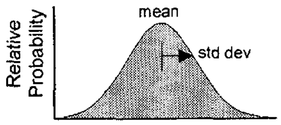
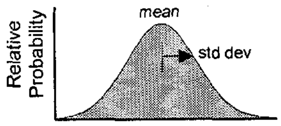
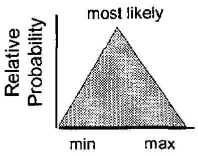
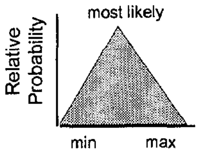
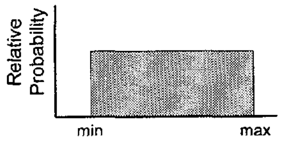
When "measured" data is not available, a triangular distribution may be used as a rough estimate of the distribution's shape. For the example problem, a triangular distribution will be assumed and the data on minimum, most likely, and maximum service live data in Table 4. 3 will be used for the distribution input parameters.

Very little is known about the variability of the discount rate in the example problem other than it ranges from 3% to 5%. When little is known about the variability of an input variable the use of a uniform distribution has the distinct advantage of giving equal probability weighting throughout the range of possible values.

For the example problem, using a uniform distribution to describe the discount rate has the added advantage of not giving unfair advantage to any particular alternative based on the influence of higher or lower discount rates. (Typically lower discount rates favor higher initial costs and lower future costs. Higher discount rates typically favor lower initial costs and higher future costs.

In the example problem a uniform distribution with a minimum of 3% and a maximum of 5% is used to describe the variability associated with the discount rate. Table 4. 4 provides a summary of input distributions that will be used in the analysis.

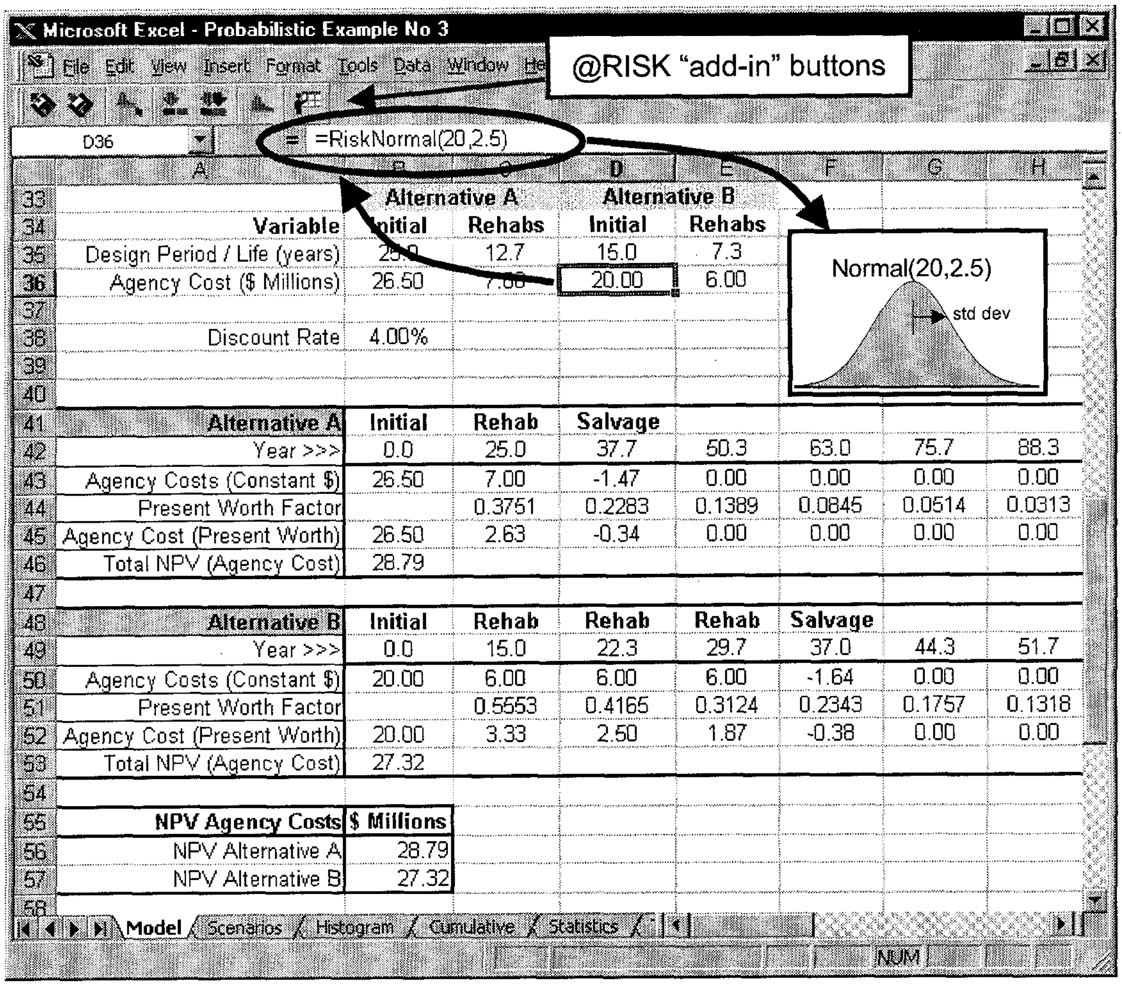
Table 4. 4 Summary of Input Distributions for LCCA.

Variable	Distribution Type	Distribution Type and Controlling Parameters	
Initial Agency Cost	Normal	Normal(mean, std dev) Alt A – Normal (26.5, 0.75) Alt B – Normal(20,2.5)	
Future Rehab Cost	Normal	Normal(mean, std dev) Alt A – Normal (7, 0.5) Alt B – Normal(6,1)	
Pavement Service Life - Initial Constr.	Triangular	Triang(min, most likely, max) Alt A – Triang(20,25,30) Alt B – Triang(12,15,18)	
Pavement Service Life – Rehabilitation	Triangular	Triang(min, most likely, max) Alt A – Triang(10,13,15) Alt B – Triang(5,7,10)	
Discount Rate	Uniform	Uniform(min, max) Alt A – Uniform(3,5) Alt B – Uniform(3,5)	

4.4.2.5 Incorporating Probability Distributions In Spreadsheet Models

Probability distributions can be directly incorporated into spreadsheet models using custom distribution functions available as part of the @RISK and Crystal Ball software programs. Each of the programs includes approximately 30 thirty new spreadsheet functions that are "add-ins" to the spreadsheet's basic function set. Each one of these 30 or so new functions represent a type of probability distribution (NORMAL, BETA, TRIANG, etc.).

Figure 4. 7 shows a Microsoft Excel/@RISK model of the example problem. It illustrates how the initial agency cost for alternative B (cell D36) can be programmed as an @Risk distribution function by entering both the function name for the distribution type desired, (such as RiskNormal for a Normal distribution) and the arguments which describe the shape and range of the distribution, (such as RiskNormal(20,2.5) for a $20 million average value, and a $2.5 million standard deviation).


Microsoft Excel - Probabilistic Example No 3
File Edit View Insert Format Tools Data Window He
@RISK “add-in” buttons
Normal(20,2.5)
std dev
NUM
Model
Scenarios
Histogram
Cumulative
Statistics

D36 = =RiskNormal(20,2.5)

	A	B	C	D	E	F	G	H
33		Alternative A		Alternative B				
34	Variable	Initial	Rehabs	Initial	Rehabs			
35	Design Period / Life (years)	25.0	12.7	15.0	7.3			
36	Agency Cost ($ Millions)	26.50	7.00	20.00	6.00			
37								
38	Discount Rate	4.00%						
39								
40								
41	**Alternative A**	**Initial**	**Rehab**	**Salvage**				
42	Year >>>	0.0	25.0	37.7	50.3	63.0	75.7	88.3
43	Agency Costs (Constant $)	26.50	7.00	-1.47	0.00	0.00	0.00	0.00
44	Present Worth Factor		0.3751	0.2283	0.1389	0.0845	0.0514	0.0313
45	Agency Cost (Present Worth)	26.50	2.63	-0.34	0.00	0.00	0.00	0.00
46	Total NPV (Agency Cost)	28.79						
47								
48	**Alternative B**	**Initial**	**Rehab**	**Rehab**	**Rehab**	**Salvage**		
49	Year >>>	0.0	15.0	22.3	29.7	37.0	44.3	51.7
50	Agency Costs (Constant $)	20.00	6.00	6.00	6.00	-1.64	0.00	0.00
51	Present Worth Factor		0.5553	0.4165	0.3124	0.2343	0.1757	0.1318
52	Agency Cost (Present Worth)	20.00	3.33	2.50	1.87	-0.38	0.00	0.00
53	Total NPV (Agency Cost)	27.32						
54								
55	**NPV Agency Costs**	**$ Millions**						
56	NPV Alternative A	28.79						
57	NPV Alternative B	27.32						
58								

Figure 4. 7 Excel Spreadsheet Showing @RISK “add-in” Buttons.

Distribution functions such as this may be used anywhere in the spreadsheet where there is uncertainty about the value of the variable. @RISK functions may be used the same way that any of the other normal spreadsheet functions are used. They may be included in mathematical expressions, have cell references or formulas as arguments, be used as part of IF–THEN expressions, etc.

4.4.3 Perform Simulation

After the structure of the model has been established and distributions developed and substitued for the uncertain input variables, the next step is to actually run a simulation of the model to obtain results. A simulation is essentially a rigorous extension of a sensitivity analysis whereby different randomly selected sets of values from input probability distributions are used to calculate results which are then arrayed in the form of a distribution of all possible outcomes. This process of using random numbers to sample from probability distributions is known as the Monte Carlo.(9) The process of sampling from an input distribution is depicted in Figure 4. 8.

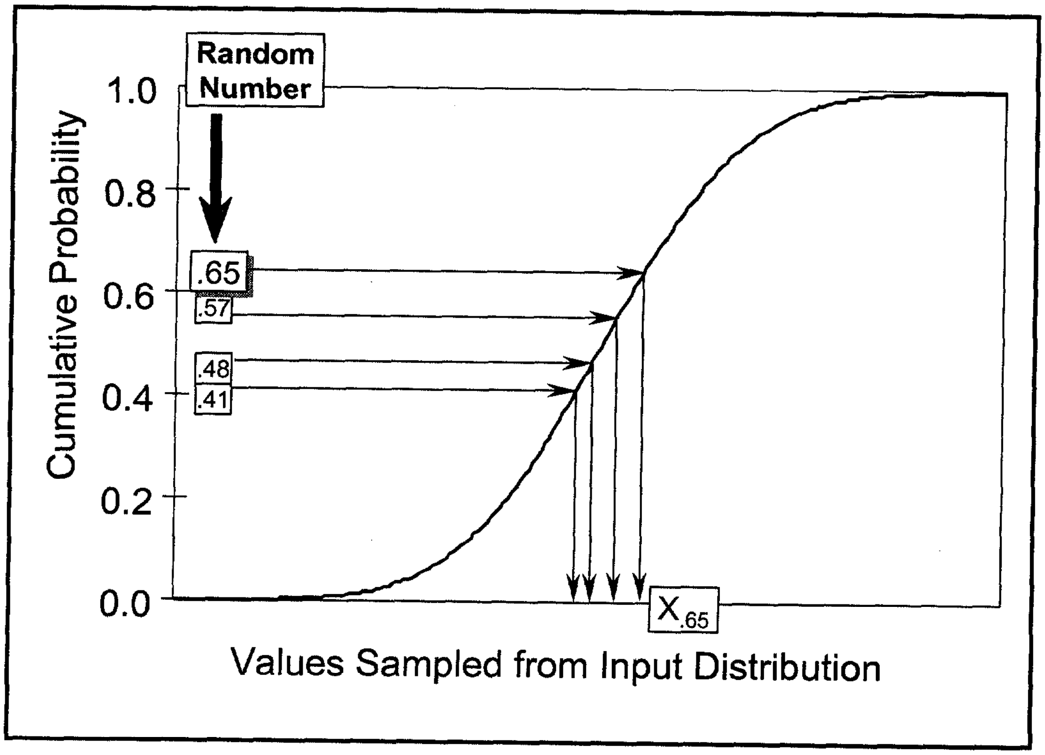


Figure 4. 8 Monte Carlo Sampling Showing Four Iterations (10).

Figure 4. 8 shows an ascending cumulative probability distribution that describes the range and probability of occurrence of values for a given uncertain variable. The figure also shows a series of random numbers between 0 and 1 along the probability scale of the cumulative distribution that were generated by the computer in a Monte Carlo simulation. The corresponding values sampled on the x scale are also shown. For example, when the computer generates the random number 0.65 in Figure 4. 8, a corresponding value of $x_{.65}$ is sampled. All values along the cumulative scale on the y-axis have equal probability of being selected. However, X axis values corresponding to portions of the distribution curve where the slope is steeper (more vertical) have a greater likelihood of being sampled compared to x axis values that correspond to portions of the curve with flatter slopes.

In real simulations, each iteration represents a possible scenario or outcome. The results of each iteration are captured, compiled and subjected to rigorous statistical analysis. This process of sampling from a probability distribution is repeated over and over until the specified number of iterations are completed or until the simulation process reaches convergence (i.e. the point at which additional iterations do not significantly change the distribution of the computed output).

Simulation techniques, such as the Monte Carlo, typically require a large number of iterations in order to ensure that there is sufficient opportunity to sample values with low probabilities of occurring. This is especially true when highly skewed distributions are used to describe the input variables. When the number of iterations performed is too low, "clustering" can occur. In such situations, sampled values are tightly clustered around high probability outcomes and the low probability outcomes necessary to flesh out the full range of possible outcomes are not represented.) This facet is particularly important when low probability outcomes can have an inordinately serious impact on results. Such outcomes must be accounted for in the simulation, but, to do this, they must be sampled. This has led to the development of a sampling technique known as Latin Hypercube.(11,12,13) Latin Hypercube forces a more representative sampling with a much lower number of iterations.

Latin Hypercube is a stratified sampling technique where the probability scale of the cumulative distribution curve is divided into a number of equal probability ranges. The number of ranges used is directly dependent on (equal to) the number of iterations performed in the simulation.

Figure 4. 9 illustrates the concept. Figure 4. 9 shows four iterations and therefore four distinct divisions of the cumulative scale. Within a division, as shown in Figure 4. 9 division number 2, a random number between 0 and 1 is generated and serves as the basis for selecting the input value. In the case shown 0.4 is generated and a corresponding value is sampled from the distribution. In future iterations, the previously sampled section is not sampled again. In other words, the Latin Hypercube technique, "memorizes" the sections it sampled and therefore achieves convergence much more quickly compared to the Monte Carlo. In addition to being more efficient this technique ensures low probability values are sampled and included in the simulation.

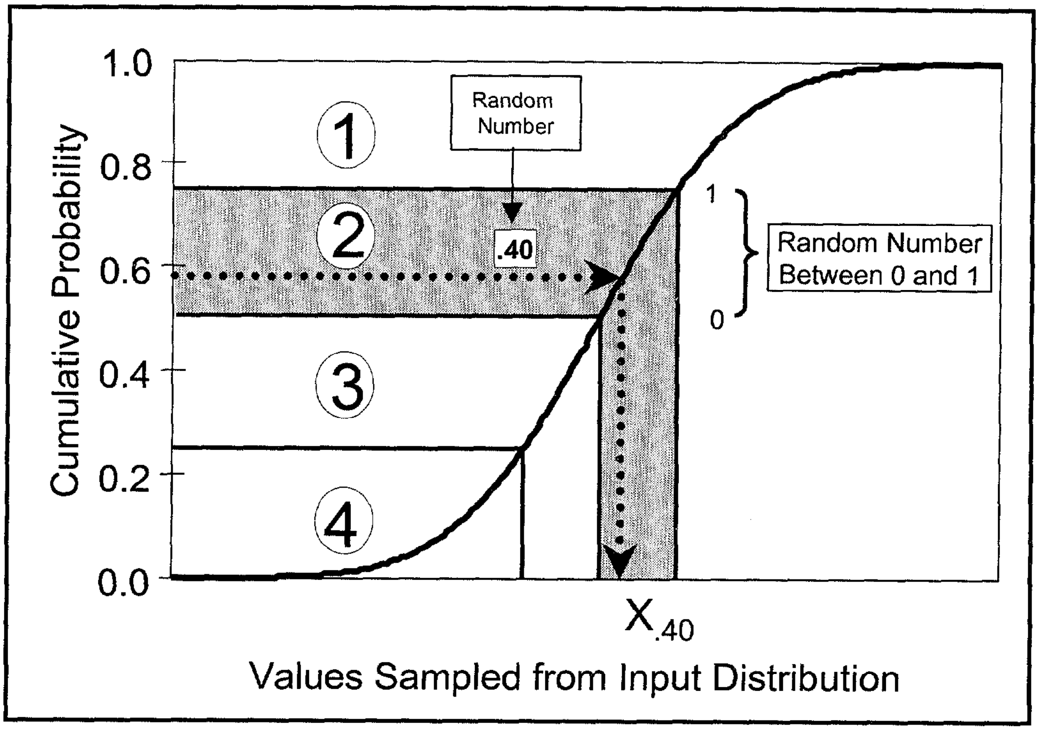

1.0
0.8
0.6
0.4
0.2
0.0
Cumulative Probability
Random Number
.40
①
②
③
④
1
0
Random Number Between 0 and 1
$X_{.40}$
Values Sampled from Input Distribution

Figure 4. 9 Latin Hypercube Sampling Showing Four Iterations (14).

Figure 4.10 and Figure 4.11 compare the effect of Monte Carlo and Latin Hypercube sampling when the number of iterations are held to 100. As shown, 100 samples were randomly drawn from the normal distribution function for Initial Agency Cost Alternative B - Normal(20, 2.5). As shown in Figure 4.11, the stratified Latin Hypercube sampling technique achieves a distribution that far more resembles the familiar bell shaped normal curve compared to pure random Monte Carlo sampling. Because of stratified sampling it is possible to achieve convergence in fewer iterations compared to the Monte Carlo.

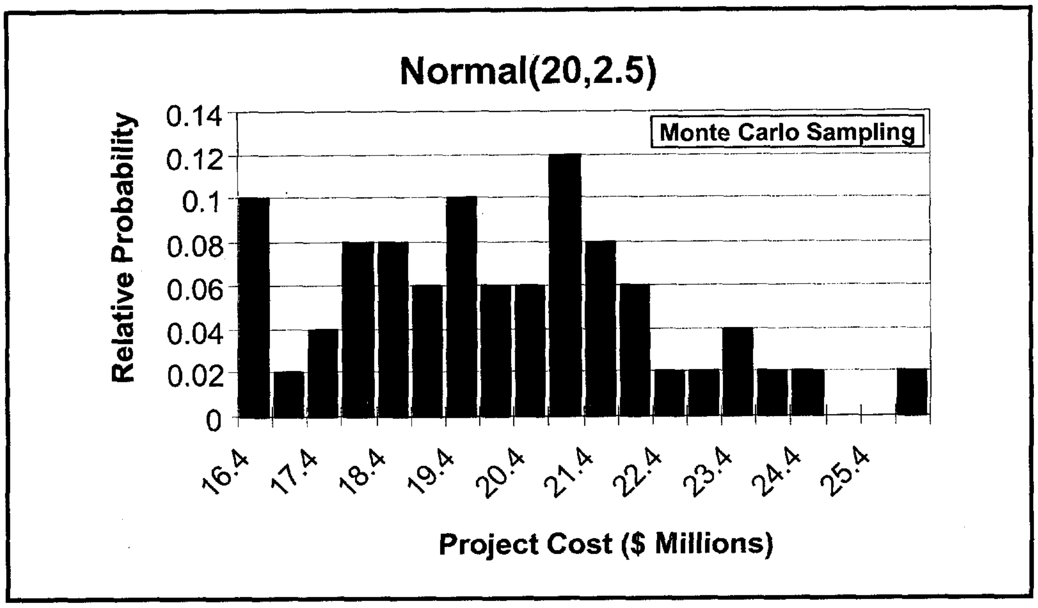


Figure 4.10 Monte Carlo Sampling – One Hundred Iterations.

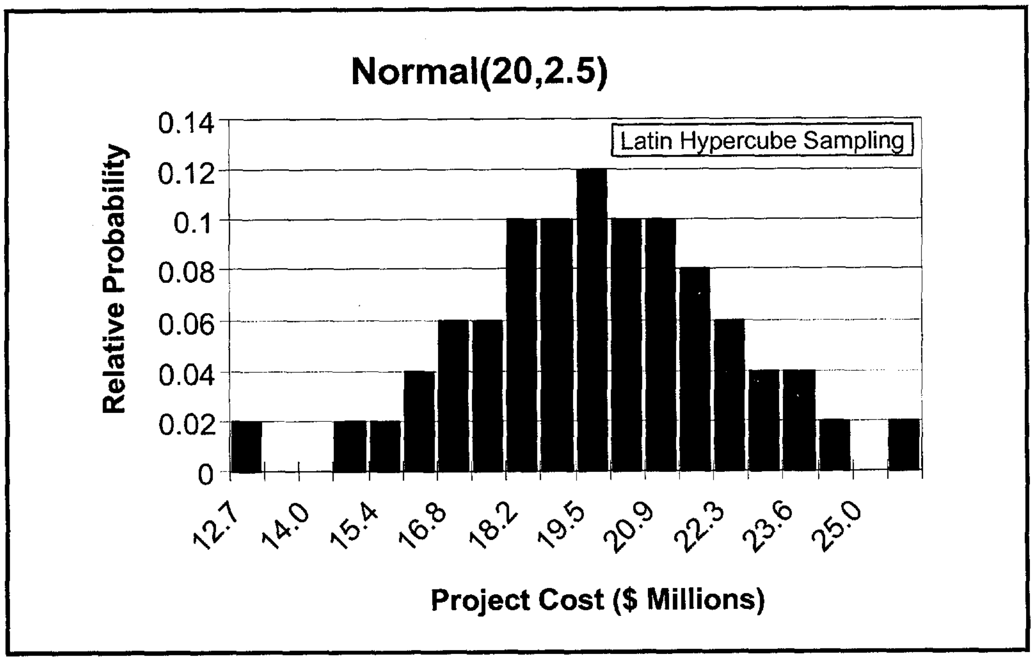


Figure 4.11 Latin Hypercube Sampling – One Hundred Iterations.

A cardinal rule of risk analysis modeling is *every iteration of a risk analysis model must be a scenario that could physically occur.* Failure to recognize the relationship between input variables is usually a prime source of error in simulation modeling that will produce misleading results. For example, suppose as part of a risk model the random variability of annual average daily traffic (AADT) and pavement design thickness were described by probability distributions. Typically, pavement engineers design thick pavements for high traffic volumes to limit strain at the base of the pavement structure. Therefore, it would be irrational to allow a simulation model to sample the traffic probability distribution curve for high values of AADT at the same time it sampled the design thickness probability distribution curve for low values of pavement thickness. Formal treatment of this relationship between AADT and pavement design thickness is usually accounted for in the risk model using a correlation matrix. Given that user costs are omitted from the example problem the number of variables has been greatly reduced and we do not have to correlate variables.

Ten thousand iterations were processed using the Microsoft Excel @RISK model shown in Figure 4. 7. The run time for this simulation was 47 seconds using a Pentium 166 MHz Computer with 80 MB Ram.

Before analyzing the simulation results it is important to stretch the assumptions to make certain the model is robust across the entire range of possible values for the uncertain inputs. In other words, the model should be tested as each component is added to make certain the results are reasonable. A visual tool to determine what is happening during a simulation is to construct a plot of the performance curves and cash flows. Some risk software, such as @RISK, will allow you to run a simulation one iteration at a time. By running the simulation in "slow motion – one sample at a time" and viewing the changing shape of the output graphs in real time it is possible to detect errors that may exist in the model. If the model is large, a test and build approach is recommended.

Another tool that may be used to test a risk model is to incorporate the use of a random number generator seed. A seed value is a value that is used to start the sampling from input distributions and all subsequent random numbers will rely on this value. Providing that the model is not changed, a second run of the simulation using the same seed value will produce exactly the same results. This can be very useful in testing the effect of varying distributions on risk results. By using a seed value one is certain that any change in the result is due to changes in the model and not a result of randomness of the sampling. After you are sure the model is producing results that are correct, then you may proceed with analyzing and interpreting the results.

4.4.4 Analyze and Interpret Results

If the analysis had been conducted according to the traditional "deterministic" approach all we would have to base our decision on would be the means of the input distributions: Alternative A $28.93 million and Alternative B $27.60 million, respectively. From analysis of the NPVs we can readily see that Alternative B is less than Alternative A by $1.33 million (~ 4.8 percent). Most State highway agencies would probably consider the difference in means insignificant and may exclude lowest NPV as the basis for selecting the alternative. Without an analysis of the variability about the mean a decision such as this may prove to be a poor choice - depending on the decisionmaker's tolerance for risk.

Interpretation of risk analysis results go beyond a simple comparison of which alternative cost less to include an analysis of the likelihood that any particular outcome will occur. There is no presumption that any particular alternative is correct. Instead, as shown in Figure 4.12, the whole range of conceivable outcomes are arrayed together with the estimated probability of each occurring . Figure 4.12 shows the risk profile of the Net Present Value for Alternatives A and B in histogram form where the probability is the area under the curve. The main advantage of the histogram is that it readily shows the variability about the mean. The wider the distribution the greater the variability. As shown the outcome for Alternative B is more uncertain than Alternative A.

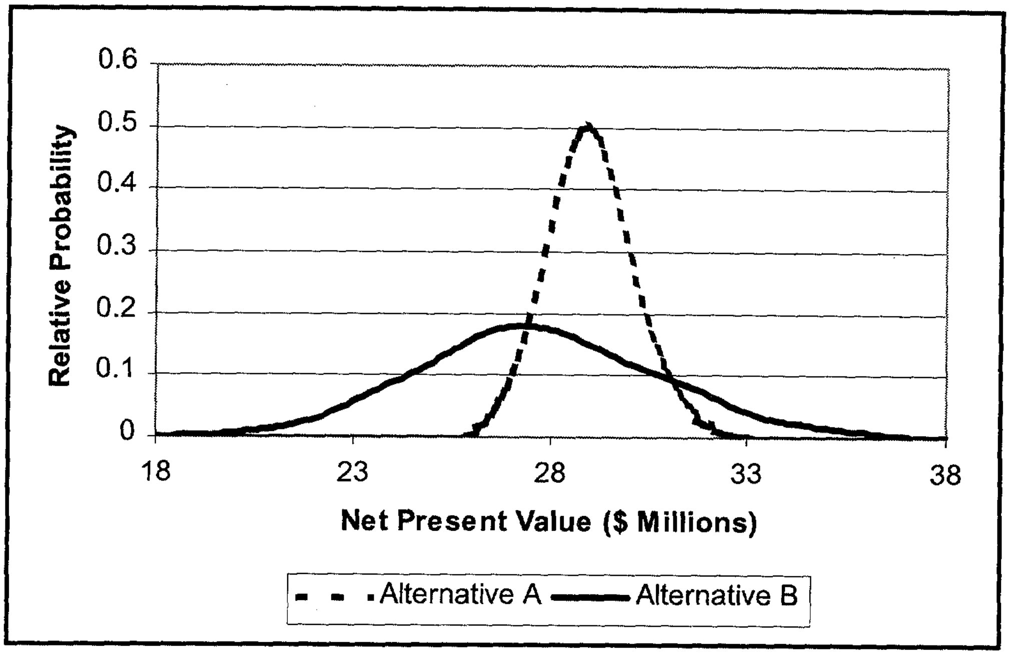


Figure 4.12 Histogram Risk Profile of Net Present Value for Alternatives A and B.

In interpreting the risk profile in Figure 4.12 it is important to distinguish between downside risk and upside risk. Upside risk for project cost implies cost overrun - chance of financial failure. Downside risk for project costs implies cost underrun - chance for profit. As shown in Figure 4.12 Alternative B has greater downside risk compared to Alternative A. However, as discussed below it is important to quantify the probability of cost overrun for Alternative B.

As shown in Figure 4.13 risk profiles may be presented in the cumulative form. As shown in the figure there is a 60% relative probability that project costs for Alternative B will be less than $28.27 million. This means that for the 10,000 iterations that were processed, 60% of the calculated values for NPV were less than $28.27 million. The variability for the proposed alternative is inversely proportional to the slope of the cumulative curve. In other words, the steeper the slope the less variability. The flatter the slope the greater the variability. As shown Alternative B is more "flatter" than Alternative A and is therefore more variable.

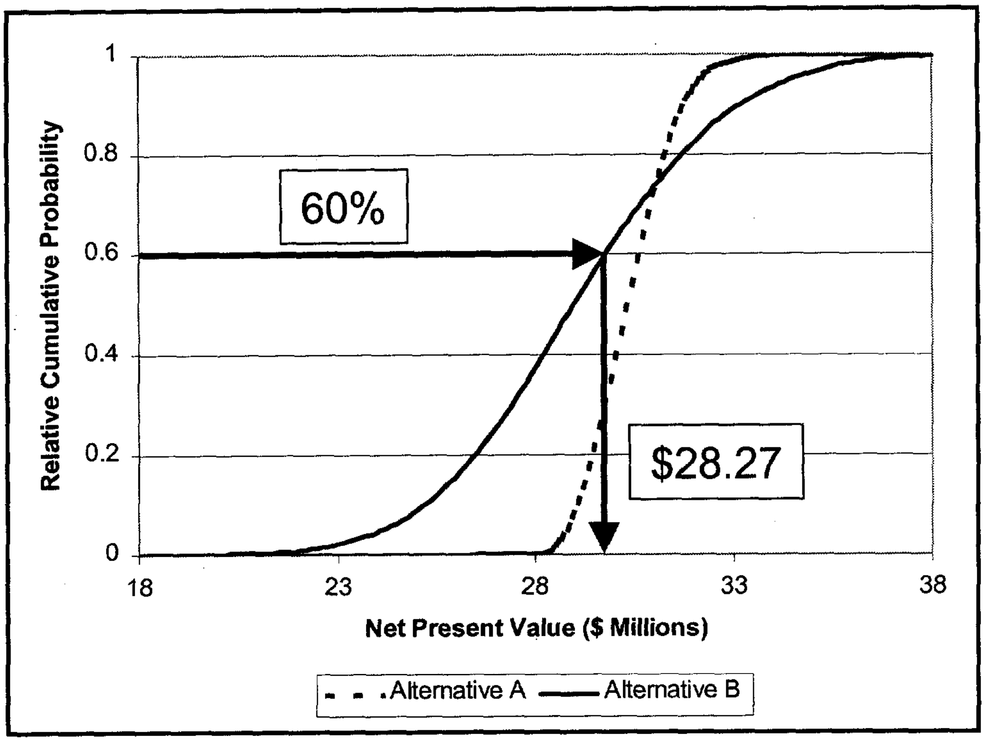


Figure 4.13 Cumulative Risk Profile of Net Present Value for Alternatives A and B.

A risk analysis provides much more information than a simple deterministic solution. As shown in Table 4. 5 additional information comes in the form of basic statistical measures of simulation results that clarify the uncertainty associated with each alternative. In interpreting the risk involved with each alternative it is important to identify the magnitude of the extremes of the distributions shown in Figure 4.12. As shown in Table 4. 5

> Alternatives A and B have minimums of $25.4 and $13.3 million and maximums of $33.04 and $40.35 million, respectively.
>
> The standard deviation for Alternative B is greater than Alternative A.

Analysis of the tails of the distributions reveals:

> A 10% probability that the NPV of Alternative B will be less than Alternative A by as much as $3.94 million. On the other hand there is
>
> A 10% probability that Alternative B will exceed the cost of Alternative A by $1.35 M.
>
> However, 70% of the time the NPV for Alternative B was less than Alternative A.

Table 4. 5 Risk Profile Basic Statistics for Alternatives A and B.

	Net Present Value ($ Millions)		
Basic Statistic	**Alternative A**	**Alternative B**	**B – A**
Minimum	$ 25.40	$ 13.13	$ (12.27)
Maximum	$ 33.04	$ 40.35	$ 7.31
Mean	$ 28.93	$ 27.60	$ (1.33)
Std Deviation	$ 1.04	$ 3.13	$ 2.09
Percentile			
5%	$ 27.33	$ 22.66	$ (4.67)
10%	$ 27.65	$ 23.71	$ (3.94)
15%	$ 27.88	$ 24.39	$ (3.49)
20%	$ 28.05	$ 24.98	$ (3.07)
25%	$ 28.22	$ 25.50	$ (2.72)
30%	$ 28.36	$ 25.93	$ (2.43)
35%	$ 28.49	$ 26.34	$ (2.15)
40%	$ 28.62	$ 26.73	$ (1.89)
45%	$ 28.75	$ 27.12	$ (1.62)
50%	$ 28.87	$ 27.48	$ (1.40)
55%	$ 29.00	$ 27.87	$ (1.13)
60%	$ 29.13	$ 28.27	$ (0.86)
65%	$ 29.27	$ 28.71	$ (0.56)
70%	$ 29.42	$ 29.13	$ (0.29)
75%	$ 29.58	$ 29.63	$ 0.06
80%	$ 29.76	$ 30.20	$ 0.44
85%	$ 30.01	$ 30.88	$ 0.87
90%	$ 30.31	$ 31.67	$ 1.35
95%	$ 30.75	$ 32.98	$ 2.23

As part of the risk assessment a sensitivity analysis may be performed on simulation results to identify the input distributions which are most important in determining the output distributions. The results of this analysis are usually displayed in the form of a Tornado plot as shown in Figure 4.14 and Figure 4.15. The higher the correlation coefficient the more significant the input variable is on determining the output's value. The variables listed at the top of the graph are more significant than those at the bottom. The degree of correlation may be calculated using either the rank order correlation or stepwise least squares regression. Rank order correlation makes no assumption about the relationship between the input and output variables. Least squares regression assumes a linear relationship between input and output variables. This is important to note since models that incorporate divisions and power functions often violate this later assumption. Figure 4.14 and Figure 4.15 both use rank order correlation. Typically, correlation coefficients less than 0.6 are not significant.

As shown in Figure 4.14 Initial Agency Cost has a correlation coefficient of 0.72. What does this mean? This tells us that if Initial Agency Cost moves one standard deviation (in either direction), then we can expect the NPV for Alternative A to move 0.72 of a standard deviation in the same direction. If Design Period / Life Initial Construction moves one standard deviation (in either direction), then we can expect the NPV for Alternative A to move 0.52 standard deviations in the opposite direction, since the relationship is reversed.

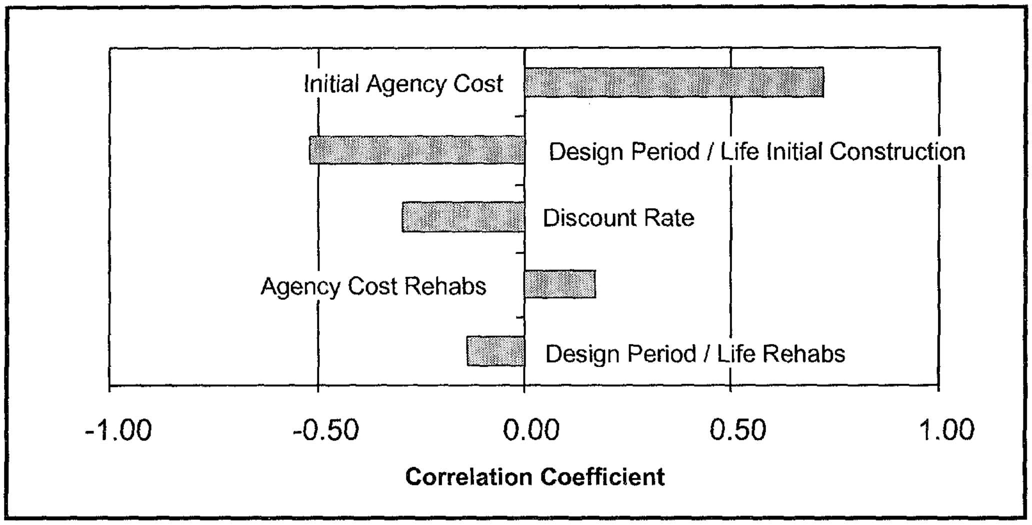


Figure 4.14 Correlation Sensitivity Plot for NPV Alternative A.

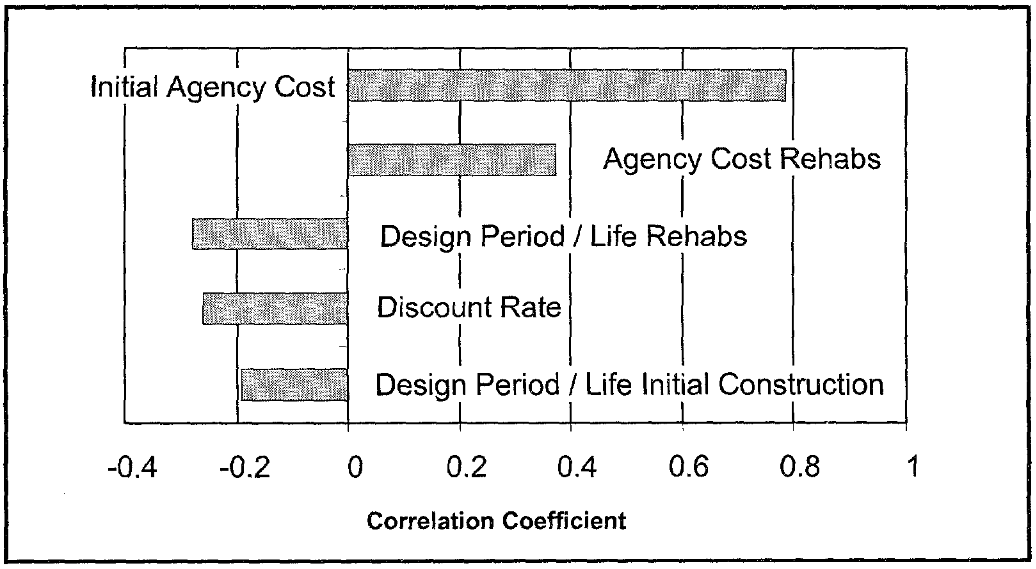


Figure 4.15 Correlation Sensitivity Plot for NPV Alternative B.

To further our analysis, we may want to explore the low probability area of the outcome distribution curve for Alternative B, where the NPV is greater than that for Alternative A, greater than the 90th percentile. As shown in Figure 4.16 what are the variables that produce the low probability scenarios – the extremes, or tails, of the results distribution for Alternative B?

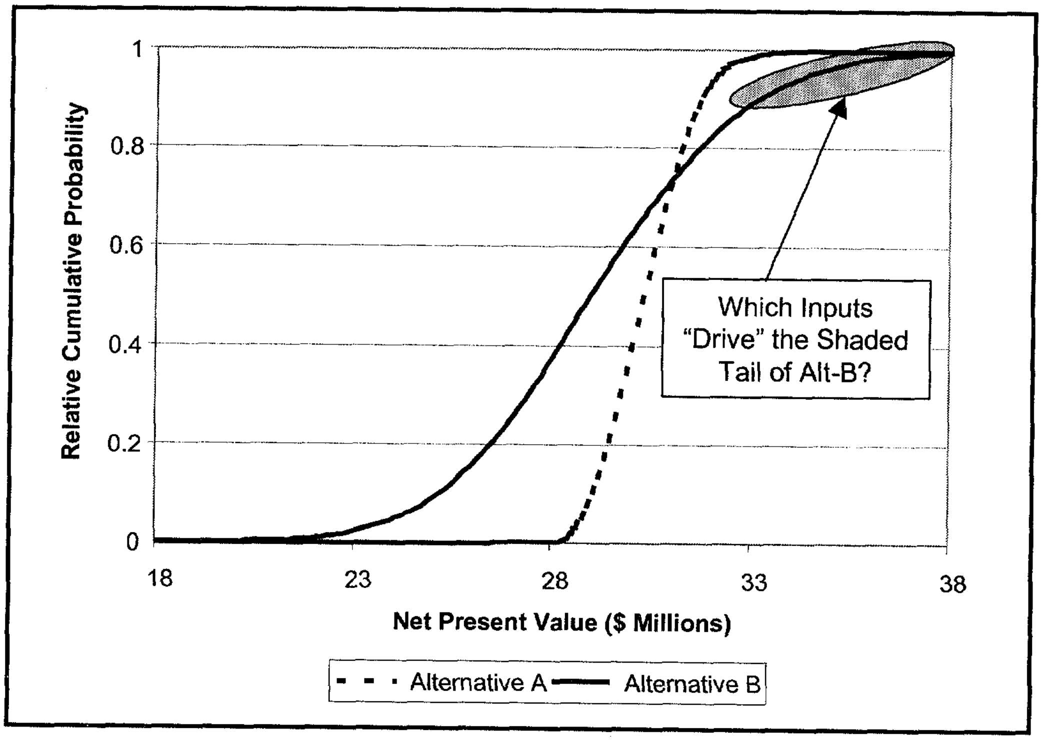


Figure 4.16 Analysis of Distribution Tails.

This type of "scenario analysis" is important to identify those events that may cause a project to have significant cost overrun. The analysis is easily accomplished using Excel/@RISK which uses the following procedure to identify "significant" inputs.

1. Each input variable that affects the selected output is found.
2. The median and standard deviation of each input is calculated.
3. A 'subset' is created containing only the iterations in which the output achieves the defined target.
4. The median of each input found in step 1 is calculated for the subset data.
5. For each input found in step 1, the difference between the simulation median and the subset median is calculated and compared to the standard deviation of the input data. If the

absolute value of the difference in medians is greater than 0.5 of the standard deviation of the whole, then the input variable is deemed 'significant' - otherwise the input is ignored.

As shown in Table 4. 6 when NPV for Alternative B is greater than the 90th percentile significant inputs are Design Period / Life Initial Construction, Agency Cost Initial Construction, and Agency Cost Rehabs. Now that the "drivers" in the tail are known the decisionmaker may choose to take some mitigating action against these significant inputs to reduce exposure to upside risk. For example, the decisionmaker may choose to ensure that Agency Costs for Initial construction and rehabilitation comes in below the median values shown to reduce risk of significant cost overrun.

Table 4. 6 Scenario Analysis Results for NPV Alternative B.

Significant Inputs When NPV Alternative B Greater Than 90th Percentile	**Median of Samples in Subset Iterations Meeting Target**
Design Period / Life Initial Construction (Years)	14.1
Agency Cost Initial Construction ($ Millions)	21.8
Agency Cost Rehabs. ($ Millions)	5.43

4.4.5 Make Consensus Decision

In our problem we have two competing alternatives, each with varying levels of risk. In order to make a decision based on risk analysis results it is important for the decisionmaker to define the level of risk the organization can tolerate. Decisionmakers that can tolerate little risk prefer a small spread in possible results, with most of the probability associated with desirable results. On the other hand, if the decisionmaker is a risk-taker, then he or she will accept a greater amount of spread, or possible variation in the outcome distribution. After weighing the probability for upside risk, as well as downside risk, and identifying the variables that drive the low probability tails most decisionmakers can reach a decision. In our example, clearly Alternative B appears to be the better alternative since there is far greater likelihood of cost savings compared to Alternative A. Also, the probability of cost overrun, compared to Alternative A, appears to be quite low – less than 30%.

Why does this step state that the decision is a consensus? This is because early in the process (Step 2) all stakeholders were involved in developing the input distributions. As a result, this overall process should diffuse any negative reaction special interest groups may have with the outcome. The end result is that the entire risk analysis process facilitates consensus building among stakeholders so that action, in the best public interest, may be taken.

4.5 PRESENTING RISK ANALYSIS RESULTS

A risk analysis is of little value if the results cannot be understood. The purpose of this section is to offer practical advice in presenting risk analysis results to decisionmakers. The first step in presenting risk analysis results is to know your audience. Here are some basic questions you should ask prior to presenting your results.

☐ Does your audience need a risk primer?
☐ Does your audience buy-in to the risk analysis approach?
☐ Does your audience buy-in to your analysis?

Does your audience need a risk primer? One of the primary reasons people have difficulty interpreting risk analysis results is they often have little understanding of basic statistics which is a prerequisite to interpreting risk results. Therefore, it may be necessary to provide an introductory session to train decisionmakers on fundamental risk analysis concepts such as Monte Carlo sampling and probability distributions prior to presentation of risk analysis results. The purpose of such a session is to provide just enough information so the decisionmaker can understand and interpret risk analysis results. There is no reason to include a formal treatment of the statistical theory underlying the concepts.

Does your audience buy-in to the risk analysis approach? Although Monte Carlo simulation techniques have been around since the early 1940's its only recent that it has been employed in the spreadsheet world of the analyst. Risk analysis is therefore a relatively new way of treating uncertainty and involves an entirely new approach to understand and interpret the results. Risk analysis is an analytical tool to aid the decisionmaker with the decision to select the best alternative. By exposing previously hidden areas of uncertainty critical information is revealed thereby providing the decisionmaker the opportunity to take mitigating action to decrease exposure to risk. Most importantly, risk analysis provides those vested with the appropriate authority, namely executives and elected official, the opportunity to make decisions about risk taking. As a compliment to LCCA, risk analysis elevates the debate, from the validity of LCCA results, to taking action that is in the best interest of the general public.

Does your audience buy-in to your analysis? Anyone examining risk analysis results should be highly skeptical. Serious questions should be asked. First and foremost is your model correct? There is nothing worse than making high stakes decisions based on a model that is wrong. It is critically important that the structure and logic of the model be verified by an independent party. Particularly, if risk analysis becomes an integral part of the decisionmaking process.

Does your model follow generally accepted procedures? For example, there are a number of different procedures available to determine user costs? Are your procedures documented and do stakeholders, in general, accept the procedures. This is important to obtain buy-in on risk results. How did you determine your input distributions? Were stakeholders involved in the process? Did you use hard data to develop the distributions? Does the range of your distributions fall within a generally accepted range?

In general, the level of detail in a risk analysis report should be commensurate with the problem at hand. The presenter of a risk analysis report should be able to answer key questions regarding the risk analysis results. In the presentation it is recommended that the developer of the risk model be at hand to answer key questions about the structure and logic of the model. In the presentation of risk analysis results it is important not to overburden the audience with statistics. A simple one page summary to include summary statistics, histogram and cumulative distributions, and perhaps a Tornado graph to show the important inputs in the analysis should be sufficient. When conveying probability graphs to upper management it is highly recommended that either cumulative ascending or cumulative descending graphs be presented.

The presenter should choose a method of presentation and stay with it. Structure and logic charts are also quite useful in presenting the simulation process as well as discussing judgements about the model relationships. Model assumptions should be placed at the back of the report. These are important, but not so important as to be in the summary sheet.

4.6 SUMMARY

The Federal Government's Office of Management and Budget recommends the use of a risk analysis approach wherever possible in the treatment of uncertainty.(15) Risk analysis is a technique that exposes areas of uncertainty, typically hidden in the traditional "deterministic," approach to LCCA, and allows the decisionmaker to weigh the probability of the outcome actually occurring. Risk analysis is a technique that combines probability descriptions of uncertain variables and computer simulation to characterize risk. The chapter outlined a general approach to incorporate risk analysis into LCCA in pavement design. The chapter introduced the use of spreadsheet based Monte Carlo simulation as the premier analytical technique to incorporate the variability of inputs into the analysis.

CHAPTER 5 SUMMARY AND RECOMMENDATIONS

LCCA is an analysis technique that builds on the well-founded principles of economic analysis with a stronger focus on longer-term implications. It incorporates initial and discounted long-term agency, user, and other relevant costs over the life of alternative investments. It attempts to identify the best value (the lowest long term cost that satisfies the performance objective being sought) for investment expenditures. It is a tool to evaluate the over all long-term economic efficiency between competing alternative investment options. It does not address equity issues.

The NHS Designation Act of 1995 specifically requires States to conduct life cycle cost analysis on NHS projects costing $25 million or more. The Congressional Conference Committee reports supporting the NHS legislation make it clear that life cycle cost analysis cannot be required for other than high cost useable NHS project segments. Further, it prohibits prescriptive LCCA procedures beyond the fundamental requirements included in the NHS Designation Act LCCA definition. Executive Directors Anthony Kane's 4/19/96 Memorandum to FHWA Regional administrators provides implementing Guidance. The FHWA policy on LCCA is further defined in the FHWA's Interim Policy Statement on LCCA published in the July 11, 1994, Federal Register. A final LCCA Policy Statement was also published on September 18, 1996, on page 49187 of the Federal Register.

LCCA is a decision support tool, and the results of LCCA are not decisions in and of themselves. The LCCA results can be less important than the logical analytical evaluation framework that life cycle cost analyses foster. LCCA, while only officially mandated in a very limited number of situations, is encouraged and should be employed in analyzing all major investment decisions where such analyses are likely to increase the efficiency and effectiveness of investment decisions whether or not they meet specific NHS requirements.

LCC analysis should be conducted as early in the project development cycle as appropriate. For pavement design, the appropriate time for conducting the LCCA is during the project design stage. The LCCA level of detail should be consistent with the level of investment. Typical LCCA models based on primary pavement management strategies can be used to reduce unnecessarily repetitive analysis. Inclusion of all potential LCCA factors in every analysis is counterproductive, however, all LCCA factors and assumptions should be addressed, even if only limited to an explanation of the rational for not including eliminated factors in detail.

"Net Present Value" (NPV) is the economic efficiency Indicator of choice. The Uniform Equivalent Annual Cost indicator is also acceptable, but should be derived from NPV. Computation of B/C ratios are generally not recommended due to the difficulty in sorting out cost and benefits for use in the B/C rations.

Future cost and benefit streams should be estimated in constant dollars and discounted to the present using a real discount rate. Although nominal dollars can be used with nominal discount rates, use of real/constant dollars and real discount rates eliminates the need to estimate and include an inflation premium. In any given LCCA, real/constant or nominal dollars must not be mixed (i.e. all costs must be in real dollars or all costs must be in nominal dollars). Further, the discount rate selected must be consistent with the dollar type used (i.e. use real cost and real discount rates or nominal cost and nominal discount rates.

The discount rates employed in LCCA should be reasonable, reflecting historical trends over long periods of time. A 4% real discount rate is recommended. However, a 3 to 5 % real discount rate range or a 4% mean with 0.5% Standard deviation (which represents a Coefficient of Variation of approximately 12.5%) is well within an acceptable range and consistent with the annual update to Appendix A of OMB Circular A-94 for the period 1992-1996.

The LCCA analysis period selected should be sufficiently long to reflect long-term cost differences associated with reasonable design strategies. While an analysis period of at least 35 years is recommended for all pavement projects, including new or total reconstruction projects as well as rehabilitation, restoration, and resurfacing projects in FHWA's final Policy Statement on LCCA, an analysis period range of 30 to 40 years is not unreasonable.

Performance periods for individual pavement designs and rehabilitation strategies have a significant impact on analysis results. Longer performance periods for individual pavement designs require fewer rehabilitation projects and associated work zones user costs.

LCCA need only consider differential cost differences between alternatives. Costs common to all alternatives cancel out and are generally so noted in the text and not included in LCCA calculations. Sunk costs, which are irrelevant to the decision at hand, should not be included.

While most analyses include traditional agency costs, some do not fully account for the SHA engineering and construction management overhead, especially on future rehabilitations. This can be a serious over sight on short-lived rehabilitation strategies as SHAs design processes can be exceptionally long in an era of down sizing.

Routine, reactive type annual maintenance costs have only a marginal effect on NPV. They are hard to obtain, generally very small in comparison to initial construction and rehabilitation costs, and differentials between competing pavement strategies are usually very small, particularly when discounted over 30 to 40-year analysis periods.

Salvage value should be based on the remaining life of an alternative at the end of the analysis period as a prorated share of the last rehabilitation cost.

User costs are heavily influenced by current and future roadway operating characteristics. They are directly related to the current and future traffic demand, facility capacity, and the timing, duration and frequency of work zone induced capacity restrictions, as well as any circuitous mileage due to detours. Directional hourly traffic demand forecasts for the analysis year in question are essential for determining work zone user costs.

Vehicle delay and crash costs are unlikely to vary among alternative pavement designs between periods of construction, maintenance and rehabilitation operations. Although vehicle operating costs are likely to vary during periods of normal operations for different pavement design strategies, there is little research on quantifying such VOC differentials under the pavement condition levels prevailing in the USA. Analysts are advised to focus on work zone user costs.

As long as work zone capacity exceeds vehicle demand on the facility, user costs are normally manageable and represent more of an inconvenience than a serious cost to the traveling public. Under such circumstances, the roadway operates under free flow conditions and work zone user costs are dominated by user delay costs in traversing the work zone. They are a direct function of the speed differential and work zone length. Speed change costs are generally minor by comparison.

When vehicle demand on the facility exceeds work zone capacity, the facility operates under forced flow conditions and user costs can be immense. Queuing costs can account for over 90% of work zone user costs with the lions share of the cost being the delay time of crawling through long slow moving queues.

Different vehicle classes have different operating characteristics and associated operating costs. User costs should be analyzed for at least three broad vehicle classes: Passenger Vehicles, Single Unit Trucks, and Combination Trucks. User delay cost rates are probably the most contentious of all user cost inputs. While there are several different sources for the dollar value of time delay, the recommended mean values and ranges for the value of time (Aug 96 Dollars) shown in Table 5. 1 appear reasonable. It is important to note that commercial vehicles support higher values of travel time delay rates and that passenger vehicles, particularly pickup trucks, represent both commercial and non-commercial use.

Table 5. 1 Recommended Values of Travel.

Vehicle Class	**$ Value Per Vehicle Hour**	
	Value	**Range**
Passenger Vehicles	$11.58	$ 10 - $13
Single Unit Trucks	$18.54	$17 - $19
Combination Trucks	$22.31	$21 - $24

The indirect user cost of "just-in-time delivery", which is becoming more and more important, is more dependent on the uncertainty of the delay time, than the absolute amount of delay.

The variability within analyses input assumptions and estimates should be addressed. LCCA, as a minimum should include a sensitivity analysis Traditionally this has been done through sensitivity analysis associated with different discount rates or assigned value of time, normally evaluating a “best” and “worst” case scenario. Sensitivity analyses can also be applied to other cost components.

The ultimate extension of sensitivity analysis is a probabilistic approach, which allows all significant inputs to vary simultaneously. The prevailing term used in private industry for a probabilistic approach is "Risk Analysis"

FHWA advocates the use a probabilistic approach to LCCA that incorporates risk analysis of the variation within the various input assumptions, projections and estimates. Such an approach should include a "Truth in Analysis" statement that provides a probability listing of the confidence interval at which the differences in NPV of alternative are statistically significant. Further, LCCA results should be subjected to a statistical analysis to determine if the NPV differences between competing pavement alternatives are statistically significant, particularly when the differences are relatively small.

There are several microcomputer software programs currently on the market that work in conjunction with Lotus and Excel spreadsheets to do probabilistic risk analyses.

The key to the probability based analysis technique lies in quantifying the variation associated with input assumptions and estimates. Probability distributions of individual inputs are analyzed using a Monte Carlo type simulation model to develop an overall composite NPV probability distribution for each alternative.

Microcomputer software programs such as MicroBENCOST are available for conducting LCCA on routine pavement rehabilitation projects. It is an automated and improved version of the 1977 AASHTO "Red Book" on User Cost Analysis. The software program, with full documentation is available through McTrans Microcomputer Support Center at the University of Florida for about $100. QueWZ is another microcomputer software program available from McTrans (approximately $25.00) that can be used to simplify user cost computations.

Pavement Design LCCA Training is currently available through FHWA Demonstration Project No. 115, "Probabilistic LCCA in Pavement Design" (DP 115). It provides training on good/best practices in performing life cycle cost analyses for pavement design and selection purposes. It includes risk analysis of life cycle cost projections.

RESOURCE DOCUMENTS

NCHRP Report 133: Procedures for Estimating Highway User Costs, Air Pollution, and Noise Effects (1972).

NCHRP Synthesis Report 122, "Life-Cycle Cost Analysis of Pavements" (1985);

NCHRP Synthesis Report 142 "Methods of Cost-Effectiveness Analysis for Highway Projects" 88

NCHRP 7-12 - Microcomputer Evaluation of Highway User Benefits October 31, 1993

NCHRP Synthesis Project 20-5, Topic 27-12 - Road User and Mitigation Costs in Highway Pavement Projects: Final Draft, November 1997

NCHRP 12-43 "Life-Cycle Cost Analysis of Bridges" - Scheduled to be available in 1997.

NCHRP Synthesis 110 and 77 (Addresses future maintenance needs and costs)

NCHRP 1-34 Guidelines for Subsurface Drainage; Draft Final Report February, 1998

NCHRP RESEARCH

NCHRP 1-33, Methodology to improve Pavement Investment Decisions is a $500,000 36-month project, currently underway at the University of Washington under Dr. Joe Mahoney. The study is looking at truck operating cost for trucks frequently (exclusively) using particular routes with measurable roughness. Also looking at agency rehabilitation costs as a function of pavement condition using PMS profiles and computing A3R (Acceleration response to road roughness)

Project 2-18 FY '94 - Research Strategies for Improving Highway User Cost-Estimating Methodologies; Hickling, Louis, Brod

Project 2-18(2)FY '93 - Valuation of travel-Time Savings and Predictability in Congested Conditions for Highway User Cost Estimation

Project 2-18 (3) FY 94 - Development of an Innovative Highway User-Cost Estimation Procedure, Hickling, Louis, Brod]

NCHRP Project 2-18 (4) Proposal – Development and Demonstration of StratBENCOST Procedure - Hickling, Louis, Brod July 17, 1997

FHWA

LCC Symposium Report 12/94

Searching for Solutions; *A Policy Discussion Series*; Number 16, July 1996. Exploring the Application of Benefit/Cost Methodologies to Transportation Infrastructure Decision-Making. (Conference Proceedings from May 95 conference)

Executive Order 12893 January 26 1994

OMB Circular A-94 Annual Updates

LCC Interim Policy Statement - Federal Register Vol. 59 No. 131 Page 35404, July 11, 1994

LCC Final Policy Statement - Federal Register Vol. 61 No. 182 Page 49187, September 18, '96

US DOT OST Memorandum; "Departmental Guidance for the Valuation of Time in Economic Analysis.

FHWA NPDR Report

FHWA Highway Pavements Course Chapter 14-1

TRB Special Report 209 Highway Capacity Manual - Third Edition Updated 1994

FHWA Highway Statistics 1995

FHWA Traffic Monitoring Guide

FHWA Highway Economic Requirements System (HERS) Volume IV: Technical Report November 1996 National Level Version 2 DOT-VNTSC-FHWA-96-6 (see Page 8-27)

FHWA Construction Costs and Safety Impacts of Work Zone Traffic Control Strategies Volume II: Informational Guide; Publication No. FHWA-RD-89-210.

Joint Highway Research Project (FHWA/IN/JHRP-95/1 Final Report) Purdue University/Indiana DOT "An Evaluation of Lane Closure Strategies for Interstate Work Zones" - November 20, 1995 Raktim Pal and Kumares C. Sinha

Transportation Research Record 1568

Probabilistic Based MCP (Micro Computer Program) for LCCA for Asphalt Concrete Pavements U of MD - May 94 Witczak

AASHTO

AASHTO Red Book - A Manual on User Benefit Analysis of Highway and Bus Transit Improvements 1977

AASHTO LCCA Survey (Summer 1994)

Computer Software:

MicroBenCost
@ Risk
QueWZ
Crystal Ball
Pavement Evaluator 2.0
BestFit

HDM-3 and HDM-4 Modeling road User effects in HDM-4 (October 95??)
"Cost and Benefits through Bureaucratic Lens: Example of a Highway Project", Anthony Boardman, Aidan Vining, and W.G. Waters II; World Bank Publication (Analyst, Guardians, and Spenders)

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[5] Transfund Project Evaluation Manual, May 1997.

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